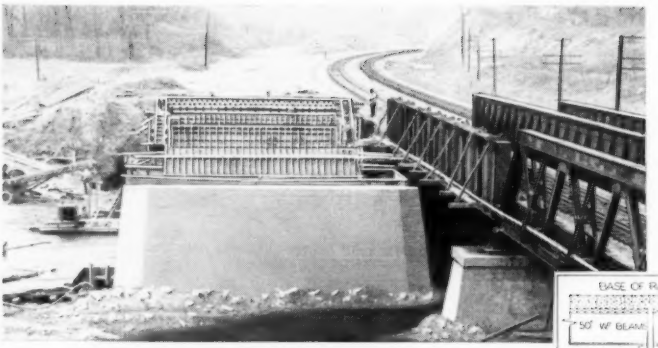
ACI News Letter

Vol. 22 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE December 1950



Scene during PREPAKT construction of Santa Fe bridge 121-A near Wilbern, Illinois.

PREPAKT REPORTS ON . . .

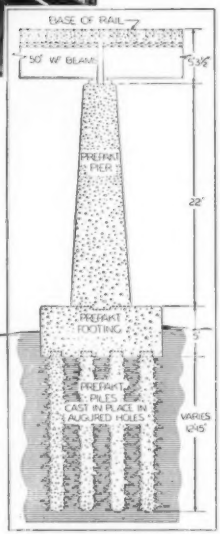
BRIDGE PIER STRUCTURE

As part of the continuous Santa Fe maintenance and modernization program, Intrusion-Prepakt recently completed the pier structures for a main line Santa Fe bridge in Illinois. Several outstanding features and methods were incorporated in the construction of the piers for this bridge.

First, the piles were cast in place in augured holes (see sketch). Extremely high load carrying capacity was provided in these piles by the double penetrating action of the coarse aggregate and natural soil penetration of PREPAKT grouting materials. These insured exceptional skin friction and high point bearing.

Next, the footings were constructed over the projecting tops of the piles, after which the pier shafts were cast on top of the footings, being "tied in" with reinforcing rods extending from the footings.

PREPAKT methods and materials thus provided these piers with exceptional strength and high resistance to deterioration from weather and other factors.



Typical Cross Section of PREPAKT pier and foundation.

HIGH RESISTANCE TO FREEZING-THAWING • LESS CRACKING
LOWER PERMEABILITY • RESISTANCE TO SULPHATE MOISTURE
HIGH COMPRESSIVE STRENGTH • HIGH BONDING STRENGTH

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December 1950

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It's worth repeating

Washington Attendance Surpasses All Previous Regional Meetings—326

ACI's Washington Regional Meeting, Washington, D. C., Oct. 24-26, surpassed all previous ACI regional meetings, as well as 14 of ACI's national conventions, in attendance figures when 326 registrants crowded into three interest-packed technical sessions.

The three-day event included three technical sessions with an inspection trip to the Chesapeake Bay Bridge now under construction concluding the conference.

The Regional Meeting Committee, under the chairmanship of E. W. Bauman, Managing Director, National Slag Assn., is to be congratulated in presenting an excellent program.

Committee members were: Frank B. Brown, Managing Director, Wire Reinforcement Institute, registration; A. N. Carter, Manager, Highway Division, Associated General Contractors, publicity; H. F. Clemmer, Engineer of Materials, Dept. of Highways, District of Columbia, entertainment; J. E. Dunn, District Engineer, Portland Cement Assn., inspection trip; A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., program; and S. S. Steinberg, Dean, College of Engineering, University of Maryland, educational contacts.

ACI President Frank H. Jackson and two past presidents, Douglas E. Parsons and Stanton Walker, served as ex officio members of the committee.

Luncheon and introduction

The conference opened with a luncheon Tuesday at which the officers, directors and special guests were introduced and the Members and guests were welcomed to Washington. The luncheon speaker was Lt.-Gen. Eugene Reybold, Executive Vice-President, American Roadbuilders Assn.

Until the voting public can be made to realize the inadequacy of our highway system, we will continue to be faced with highway and traffic problems, he emphasized.

"We must have extensions and broadening of our highway system." A continuing highway program is needed for the future, he declared. Up to date, he said, we are far behind in the nation's highway program. "We have advanced considerably, but our advances have not been adequate with the demand of motor traffic."

First Technical Session

Tuesday afternoon ACI President Frank H. Jackson, Principal Engineer of Tests, Bureau of Public Roads, served as chairman of a session that included discussions of mass concrete dams and the construction and design of pavements.

Economy plus durability

By placing 4-bag concrete on exterior faces and lean mass concrete (2½-bag mix) in the interior of dams, it is possible to achieve both durability and economy in gravity dam construction, stated Byram W. Steele, Office of Chief of Engineers, Department of the Army, at the opening session Tuesday afternoon.

He emphasized that lean mass concretes are important in minimizing volume change cracking, with the ensuing economy being an additional and attractive feature.

For many years 4 bags of cement per cu yd of concrete was considered the minimum content permissible for dam construction. "Gradually the attitude toward the 4-bag minimum for the interior of mass concrete dams began to change as the art and science of mass concrete mix design improved and the quality of concrete was brought to a higher degree of perfection," Mr. Steele said. Closer control, aggregate gradation, lower water-cement ratios and, especially, air entrainment made this possible, he continued, although there is still need for further improvement.

The entire development of lean mass concrete has been a step-by-step procedure in which the 4-bag mix has been gradually reduced to 2¼ bags. A cement content of 2¼ bags is not the minimum that can be used satisfactorily for dam interiors and still produce a structure with adequate structural, impermeable and durable qualities; the next step is a 2-bag concrete, which was successfully used in Arrowrock Dam over 35 years ago, Mr. Steel concluded.

Smooth pavements

"Smooth pavements can be built with air-entraining concrete without resorting to delayed finishing," stated Charles W. Allen, Research Engineer, Ohio Highway Dept., in discussing the precautions to be observed in finishing air-entraining concrete pavements.

In a survey of highway departments throughout the nation it was found that, in general, the placing and finishing of pavements with air-entraining concrete required no more attention than pavements built with normal portland cement, he revealed.

Assuming that the mix has been properly designed and mixed, air-entraining concrete has slightly different characteristics from normal concrete which necessitates some changes in construction technique. In this connection Mr. Allen discussed the setting of screeds in transverse finishing machines and the timing of finishing operations. He concluded by summarizing a highway department survey on the riding qualities of air-entraining concrete pavements.

Seal those pavement joints!

"Maximum service will not be maintained unless *all* joints and cracks are completely sealed to prevent infiltration of moisture and foreign materials," declared H. F. Clemmer, Engineer of Materials, Dept. of Highways, District of Columbia, in discussing concrete pavements. "The pavement slab must be made waterproof to protect the soil foundation."

"Pavement joints can be effectively sealed. It requires study and care to effect a proper seal and it requires inspection and maintenance to keep it that way. The effort and cost is necessary if we are to protect and maintain uniform soil foundations."

Mr. Clemmer, in his comprehensive report, considered the pavement design problem as related to joints and cracks. He discussed pavements designed without expansion joints and the sealing of contraction joints. The use of expansion joints in municipal streets received attention and a brief history of the development of sealing materials was presented.

Load-carrying dowels

Henri Marcus, Research Consultant, Bureau of Yards and Docks, Dept. of the Navy, reported on tests made to determine the bearing capacity of dowels when used as load-carrying devices in transverse pavement joints. He reviewed the test results in evaluating the efficiency of these devices and also considered the general problem of concrete strength.

The resistance of concrete to uniformly distributed bearing stresses was studied as was the bearing capacity of dowels loaded outside of the concrete. A third group of tests was made to study the bond resistance of coated dowels.

Oyster Bake

An informal get-together Tuesday evening in the form of an oyster bake gave all registrants a chance to meet and discuss concrete problems in general without the restraining influence of a formal presentation. The food was excellent—the discussion stimulating.

Second Technical Session

The Wednesday morning session featured an open forum on ready-mixed concrete and a brief film on construction of the prestressed Walnut Lane Bridge in Philadelphia. ACI Past President Stanton Walker, Director of Engineering, National Ready-Mixed Concrete Assn. and National Sand and Gravel Assn., presided as chairman.

Ready-mix forum

The ready-mixed concrete industry has grown from a production of about 5 million cu yd of concrete, using less than 4 percent of the portland cement produced in 1930, to a 1950 production of 30 million cu yd, utilizing about 17 percent of cement production, Chairman Walker pointed out. "Such an industry presents special problems to both producers and consumers."

The session was one of the highlights of the Washington meeting. The panel of experienced members of the ready-mix industry coped with such questions as:

What precautions are necessary when using calcium chloride as an accelerator for concrete?

What special precautions are required in handling lightweight aggregates in the ready-mixed concrete operation?

It is often required that mixer blades be replaced when unduly worn. What constitutes undue wear?

In discussing the use of calcium chloride the panel emphasized that contractors should not depend on calcium chloride to lower the freezing point of concrete. However, the intelligent use of such an admixture can be an advantage in cold weather by permitting quicker finishing and thereby lower labor costs. It was pointed out that calcium chloride seems to work more advantageously in areas with moderate winter temperatures than in the more northern areas of the United States.

The problems of handling lightweight aggregate vary with the material. It was found by one producer that it was necessary to prewet the aggregates within close tolerances; others claimed that prewetting was not required. It isn't feasible to generalize a method or a system for handling lightweight aggregates when mixing them into concrete—it must usually be done by a cut-and-try method in the field for each situation, mentioned another panel member.

In discussing blade replacement in mixers, it was found that "undue wear" varies with sections of the country, producers and specifications. In fact, a percentage of wear is not even significant in some types of blades. There is no definite measure of the amount that blades can wear and still mix concrete, but the consensus pointed to about 25 percent as a limit.

Chairman Walker pointed out that engineers have been too prone to forget that construction activities, including the mixing of concrete, retain a certain amount of know-

Where did they hail from?

Surpassing the registration figures of several ACI national conventions, the Washington Regional Meeting attracted 326 engineers, architects, builders, producers, manufacturers, etc., from as far west as Washington, as far south as Mississippi, as far north as Canada, and even two registrants from England and one from Sweden.

Calif.—2	Neb.—1
Colo.—3	N. J.—3
Del.—10	N. Y.—23
D. C.—135	Ohio—7
Ga.—1	Pa.—18
Ill.—9	R. I.—1
Ia.—3	S. C.—1
Kan.—1	Va.—23
Me.—1	Wash.—1
Md.—68	W. Va.—1
Mass.—1	Wis.—3
Mich.—3	Canada—1
Miss.—1	England—2
Mo.—2	Sweden—1

how and artisanship and specifications cannot be written that will control everything. An engineer can write specifications for mixing concrete, but someone who knows how to mix concrete must still do the job.

The panel included Alexander Foster, Jr., Vice-President, Warner Co.; A. A. Levison, Vice-President in Charge of Construction Equipment Dept., Blaw Knox Co.; Charles P. Maloney, President, Maloney Concrete Co.; Robert F. Porter, Vice-President, Harry T. Campbell Sons' Corp.; J. W. Roberts, Vice-President, Southern Materials Co., Inc.; Murray S. Simpson, Treasurer, Super Concrete Corp.; and H. F. Thomson, Manager, Redi-Mixed Concrete Division, Material Service Corp.

Third Technical Session

The presentation of technical topics continued at the Wednesday afternoon session with ACI Vice-President A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., serving as chairman.

Road Test One—MD

Already well-known to most highway engineers, Road Test One—MD has not progressed far enough to give definite answers to the problem of effects of heavy loads on concrete pavement. However, Fred Burggraf, Associate Director, Highway Research Board, described the general test program and the preliminary results on the concrete and soil tests.

The tests are being conducted on a 1.1 mile section on U. S. 301 in southern Maryland as a joint venture of several state highway departments, District of Columbia, Bureau of Public Roads and the Highway Research Board.

Production-line housing

An inexpensive method of construction and the use of easily available basic materials are prime factors in low-cost housing declared William V. Reed, Vice-President, IBEC Housing Corp., in speaking of the application of machine techniques to concrete housing construction.

In planning such construction, the corporation decided to use concrete because the basic materials are available anywhere in the world and are relatively inexpensive. Recent developments in lightweight concrete were also a factor.

The choice of concrete also permitted IBEC to achieve a flexible construction method adaptable to a wide variety of architectural, structural and climatic requirements. The method also uses a preponderance of unskilled and semi-skilled labor.

Using a mobile production line at the job site, the IBEC method produced notable economies. One 25-ton form was used for both exterior and interior walls; the form was positioned on a cast-in-place floor slab by a 30-ton crane. The roof slabs were cast on the ground in tiers and placed by a vacuum lifting device. An experiment on one project demonstrated that where speed is essential it is feasible to use the forms twice in a single day.

"Every factor in the method, from the selection of materials to design, equipment and job organization," stated Mr. Reed, "has been studied to achieve low initial cost and low maintenance cost."

Chesapeake Bay Bridge

The construction of the $37 million Chesapeake Bay Bridge was described in the paper by Robert A. Gilmore, Project Engineer, J. E. Greiner Co., and presented by George S. Jenkins.

Potomac-type pier construction was described in detail as well as other construction techniques being used on the project. The paper served as an excellent introduction to the inspection trip to the job site on the following day.

"No-fines" air-entraining concrete

"Air-gravel" concrete or "no-fines" air-entraining concrete was discussed by Rudolph C. Valore, Jr., Materials Engineer, National Bureau of Standards. Air-gravel concrete consists of high-early-strength portland cement, 20 to 30 percent of entrained air acting as fine aggregate, pea gravel and an air-entraining admixture. The properties of such concrete were discussed in detail.

The greatest problem in preparing air-gravel concrete, stated Mr. Valore, was the relative lack of reproducibility of air contents coupled with the extreme sensitivity of strength properties to air content. The com-

Continued on p. 24

Darex AEA

is a catalyzed air entraining agent, *specifically formulated* for making air entrained concrete.

Darex AEA

is being used in all types of concrete work, all over the world.
Distribution warehouse stocks are conveniently located throughout North and South America and in most foreign countries.

This air view shows base concrete being poured on the Annisquam River Bridge, Gloucester, Mass. Darex AEA concrete was used to improve workability . . to increase the life of the concrete.

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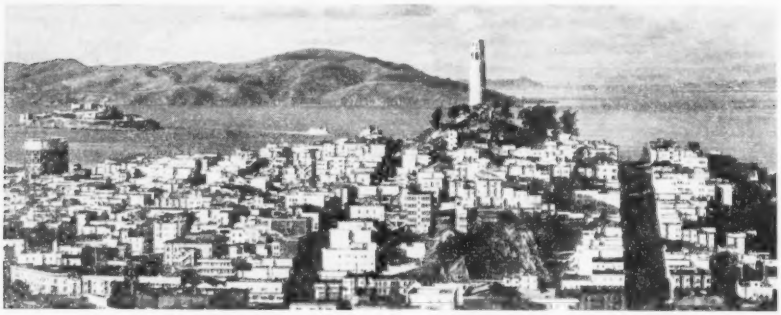

Courtesy San Francisco Convention and Tourist Bureau

Telegraph Hill

So you're going to San Francisco?

A touch of the old-world

A thick little thumb of land separated from the mainland by the 452 sq mi. of San Francisco Bay, San Francisco is only 6 mi. wide and 7 mi. long, and is completely different from any other city in the United States.

It represents a touch of the old-world and at the same time it is ultra-modern. San Francisco's foreign air, Chinatown, with its pagodas, carved temples, gilded doors, scarlet and indigo roofs and Oriental bazaars, forms the largest Chinese city outside of China.

Fisherman's Wharf is also an impressive section and you will long remember the sight of dinky cable cars mounting the straight steep inclines that are San Francisco streets.

The city, and its famed 14 hills, can be seen in all its panorama from the slopes of Twin Peaks. Telegraph Hill needs little introduction as a vantage point for views of craggy skylines and the bay area. Nob Hill, noted in the "old days" for the magnificent mansions of the bonanza kings and railroad magnates, still carries the traditions of the past although exclusive apartment houses and hotels have largely supplanted the palaces of former days. Across from Telegraph Hill lies Russian Hill, named after an ancient Russian graveyard which has long since disappeared. Highest point in San Francisco is Mt. Davidson which bears the largest concrete cross in the world on its summit.

San Francisco's ever-changing vistas give it a charm of its own—you'll want to explore it all.

Watch forthcoming News Letters for

PROGRAM PREVIEW

47th Annual Convention ★ February 20-22, 1951

See p. 22 for hotel accommodations

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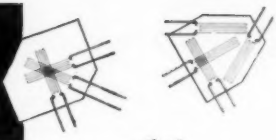

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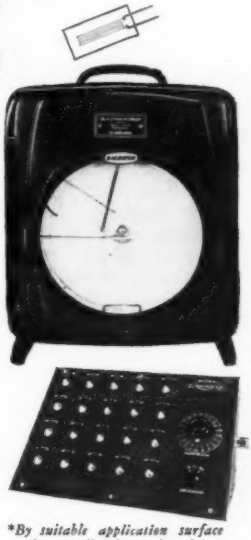
Baldwin SR-4 gages open entirely new fields in concrete testing.* Available in gage lengths from 1/16" to 6", in a variety of patterns. Recorders may be located at any convenient distance from gage. Switching unit (illustrated at bottom right) permits readings to be made on single channel instrument from a number of test points. Ask for Bulletin No. 279.



SR-4 GAGES TEST PRESTRESSED CONCRETE GIRDER. This prestressed girder, a duplicate of those which U.S. Preload Corp. will furnish for a new Philadelphia bridge, was tested to destruction under 11 times designed load. Baldwin SR-4 gages gave an accurate progress report of stresses throughout the test.

**By suitable application surface strains as well as internal strains on reinforcing bars may be determined.*

The Baldwin Locomotive Works, Philadelphia 42, Pa., U. S. A. Offices: Chicago, Cleveland, Houston, New York, Philadelphia, Pittsburgh, San Francisco, St. Louis, Washington. In Canada: Peacock Bros., Ltd., Montreal, Quebec.

TESTING HEADQUARTERS **BALDWIN**

Precasting and vacuum processing speed construction all over world

Industry does not confine its thinking within the boundaries of a single technique. In using one, it spreads out and combines several in the activities of a single project. Thus Karl Billner, Vacuum Concrete, Inc., in an expansive mood, briefed enterprises which encompass great variety: technique-wise and geographical.

Societe du Vacuum Concrete, Paris, France, is precasting 6 million sq ft of slabs 30 ft x 8 ft x 3½ in. for the Offmarsheim Canal near the border of Switzerland. The vacuum processed slabs are picked up by vacuum lifters and stacked for curing the morning after casting. A licensee, the SABLA Corp., precasts telephone cable blocks and irrigation flumes and has a contract to precast concrete members for the reconstruction of the bombed-out harbor of Bone, Algeria.

A new water reservoir for Brussels has been completed by Syndicat Vacuum Concrete, Brussels, Belgium. The steel column forms were held to the concrete floor by vacuum holders. Despite flared base and top of columns, vacuum processing through the forms permitted their removal half an hour after the concrete was placed and their re-use five times daily. Precast 12 x 12 ft ribbed roof slabs were designed for a superimposed load of 500 psf.

In Italy, Societa Italiana Vacuum Concrete, Milan, is precasting floor slabs, beams and columns and Ferro Cemento, Rome, is precasting penstocks 10 ft in diameter and 16 ft 8 in. long. Both inside and outside forms, through which the concrete is vacuum processed, are removed half an hour after they are filled with concrete.

Four-story buildings with 700 three-to five-room apartments are being precast in Cairo, Egypt, by the Heliopolis Co. Cast-in-place closure joints between beams, columns and slabs impart monolithic characteristics to the structures. One new feature is hollow precast floors requiring a minimum of concrete and steel.

In the mile-deep No. 5 shaft of the Van Dyk Consolidated Mines, Ltd., the Concrete Development Corp., Johannesburg, South Africa, is building a bisecting wall 6 in. thick at the rate of 120 ft per day using vacuum processing forms. For the Virginia Gold Mine, Ltd., this same company is precasting streamlined bracing struts reinforced with Billner interlocked steel spirals.

Large concrete pipe are precast by Vaco, Madrid, Spain, which plans to precast housing as is Inster. Ltda., Bogota, Colombia. On Guam precasting was utilized for houses, barracks, schools and power plants.

European libraries need Journals

During a recent two-month trip through Europe, Anton Tedesko, manager, structural department, Roberts and Schaefer Co., Chicago, Ill., saw much interesting work and visited European engineers and scientists interested in concrete and concrete research. All mentioned the lack of funds, especially the lack of currency which makes it impossible for them to replace books and ACI JOURNALS of past years lost during the war. Desiring to be helpful in restoring some of the important educational libraries, Mr. Tedesko has offered his help in getting back numbers of the JOURNAL to deserving libraries and engineers.

Will members who no longer have use for back numbers please *write directly* to Mr. Tedesko at the address given below listing by month, year and volume number the JOURNALS they would be willing to contribute. **Please do not send the JOURNALS.** Mr. Tedesko will reply asking for the numbers he wishes and will give instructions regarding their shipment overseas.

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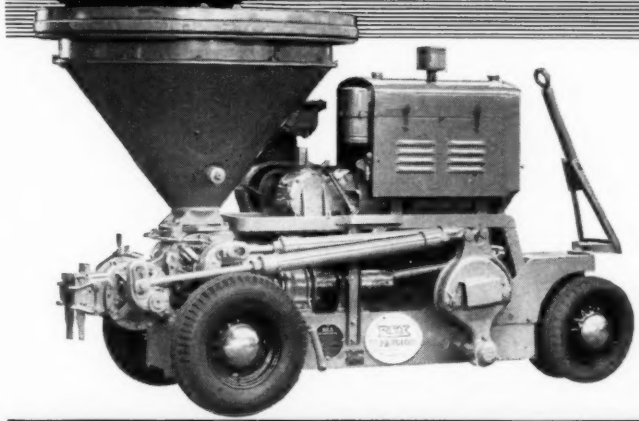

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FIRST. Where segregation of the batch has occurred through mishandling or unsatisfactory mixing, the remixing hopper of the Pumpcrete will automatically correct this condition, assuring highest quality concrete.

SECOND. When the batch contains inferior aggregates, or is a poor mix, the Pumpcrete will refuse to pump it! It *automatically* inspects batch quality!

THIRD. The piston and valves of the Pumpcrete form the plastic concrete into cylinders which are placed in the sealed pipe line and pumped directly to the forms. There is no chance for segregation of the concrete during travel through the *sealed* pipe line. Concrete arrives at the forms in the same unsegregated, high quality condition as it was when it entered the pipe line.

For highest quality concrete . . . for automatic inspection of concrete quality, rely on Rex Pumpcrete. For the complete story, write Chain Belt Company, 1713 W. Bruce Street, Milwaukee 4, Wis.

CHAIN BELT COMPANY — REX — CONSTRUCTION MACHINERY

Who's Who This Month

Committee 318

Committee 318, Standard Building Code, submits a "Proposed Revision of Building Code Requirements for Reinforced Concrete (ACI 318-47)," p. 269. Under the chairmanship of the late Chester L. Post, Washington, D. C., the committee has proposed changes in the Building Code to allow for the improved properties of new-style deformed bars complying with ASTM specification A 305-49 and the recommendations of ACI Committee 208, Bond Stress.*

C. C. Rhodes

"Curing Concrete Pavements with Membranes," p. 277, is a condensation by C. C. Rhodes, chemical research engineer, Michigan State Highway Dept. Research Laboratory, Lansing, Mich., of the basic report issued by the department on tests of membrane curing.

Mr. Rhodes' two main fields of interest are concrete and reflex-reflecting materials. He has been actively associated with the study of membrane curing materials since they were first used in Michigan and with the development of photometric tests for reflective materials.

He graduated from the University of Michigan in chemical engineering in 1921 and received an MS there in 1922. In 1939 he joined the staff of the Michigan State Highway Department's testing laboratory and 2 years later was transferred to research.

Glenway Maxon

Glenway Maxon, consulting engineer, Milwaukee, Wis., is the author of "Blade Changes Improve Tilting Mixer," p. 297.

He has been employed since 1917 in the design of construction machinery, especially that which is connected with mixing and placing concrete. President of the Cogmatic Co., Mr. Maxon has also been in consulting work since 1933. He has been connected with the development of the Dumpcrete, a nonagitating concrete truck body, the plastograph, a consistency meter, and an automatic flame cutting machine for the production of gears and sprockets.

Mr. Maxon, besides being an ACI Member, is also affiliated with the Society of Automotive Engineers, Highway Research Board, National Society of Professional Engineers and ASTM.

Gustave Magnel

Tests made prior to construction of the first prestressed bridge in the United States are described by Gustave Magnel, Professor, University of Ghent, Belgium, in "Prototype Prestressed Beam Justifies Walnut Lane Bridge Design," p. 301.

Well-known for his work in concrete structures, especially prestressed concrete, Professor Magnel directed the tests of the Walnut Lane Bridge beam as consulting engineer to Preload Enterprises, Inc., New York, the contractors for the project.

Inventor of the Magnel-Blaton system for prestressing concrete, Professor Magnel has received many honors for his pioneering in this type of construction and is the author of numerous articles and books on concrete design and construction as well as stability of structures. His most recent award was the Frank P. Brown Medal of the Franklin Institute.

Associated with the University of Ghent since 1927, Professor Magnel is director of the Laboratoire de Beton Armé at the university as well as a professor of structures and reinforced concrete. He is a member of ACI Committee 323, Prestressed Reinforced Concrete.

Myron A. Swayze

"Finishing and Curing: A Key to Durable Concrete Surfaces," by Myron A. Swayze, Director of Research, Lone Star Cement Corp., New York, N. Y., appears on p. 317.

A Wason Medalist in 1942 for noteworthy research reported in the paper "Early Concrete Volume Changes and Their Control," Mr. Swayze has long been active in ACI affairs. He is at present a member of the

Continued on p. 24

*"Report of ACI Committee 208 on Bond Stress," ACI JOURNAL, May 1950, *Proc.* V. 46, p. 677.

The size of each Bethlehem Reinforcing Bar is readily determined by means of an identifying numeral rolled into the bar. In this picture, No. 8 denotes a 1-in. round.

YOU GET

Maximum Bond

WITH THIS BAR

When you use the improved Bethlehem Reinforcing Bar, you can be sure of maximum bond strength for the life of the structure.

The Bethlehem Reinforcing Bar provides maximum bond strength because of its high, closely-spaced lugs, as called for in ASTM Specification A305-49. The anchorage these lugs make possible means low slips at working loads. This helps to prevent the formation of wide tensile cracks in the concrete, and results in better-looking, more economical structures.

The Bethlehem Reinforcing Bar is made from new-billet steel in sizes from ⅜ in. to 1¼ in. It conforms in every way to the requirements of ASTM Specification A305-49. To be sure of maximum bond, specify it for your next concreting job.

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BETHLEHEM REINFORCING BARS

Positions and Projects — ACI Members

Kerekes named Building Code chairman

Prof. Frank Kerekes, Assistant Dean, Division of Engineering, Iowa State College, Ames, Ia., has been named chairman of ACI Committee 318, Standard Building Code. He succeeds the late Chester L. Post.

Professor Kerekes has been active on ACI technical and administrative committees and is at present a member of the Technical Activities Committee. He is also chairman of the Iowa Building Code Council which recently submitted a proposed "Iowa State Building Code" to the State General Assembly.

Joint ACI-ASCE committee formed to study shear and diagonal tension

With the assignment of developing methods for designing reinforced concrete members to resist shear and diagonal tension in conformance with the new ultimate strength design methods, ACI and ASCE have formed a joint committee which will be listed as ACI Committee 326, Shear and Diagonal Tension, and as a subcommittee of ASCE's Masonry and Reinforced Concrete Committee.

Charles S. Whitney, Ammann and Whitney, Consulting Engineers, New York and Milwaukee, was appointed chairman and C. A. Willson, American Iron and Steel Institute, New York, was named secretary.

Other committee members are: Raymond Archibald, Bridge Division, Bureau of Public Roads, Washington; Boris Bresler, University of Calif., Berkeley; Arthur P. Clark, National Bureau of Standards, Washington; Alfred Freudenthal, Columbia University, New York; Eivind Hognestad, University of Illinois, Urbana; D. P. Jenny, Portland Cement Assn., Chicago; Douglas McHenry, Bureau of Reclamation, Denver; Douglas E. Parsons, Building Technology Division, National Bureau of Standards, Washington; Raymond C. Reese, Consulting Engineer, Toledo; and F. G. Thomas, Building Research Station, Dept. of Scientific and Industrial Research, England.

Janney joins Committee 323

Jack R. Janney, Assistant Engineer, Structural Development Section, Portland Cement Assn., Chicago, Ill., was recently appointed to membership on ACI Committee 323, Prestressed Reinforced Concrete.

Committee 711 adds member

Prof. O. Neil Olson, Dept. of Civil Engineering, Marquette University, Milwaukee, Wis., has accepted appointment to ACI Committee 711, Precast Floor Systems for Houses.

Franklin Institute honors Freyssinet

Eugene Freyssinet, Paris, France, famous pioneer in prestressing concrete, was awarded the Frank P. Brown Medal by the Franklin Institute recently for his development of engineering and techniques for prestressed concrete.

In announcing the award, the Franklin Institute said, "Before 1923 prestressed concrete members were not a practical reality competing economically or even technically with other structural forms. . . . It was Eugene Freyssinet of France who proved the necessity to use both high-quality concrete and high-tensile steel. . . . Once Freyssinet had established this basic principle, engineers could proceed to develop suitable methods of tensioning and anchoring stressed steel wires. . . ."

U. of Minnesota honors McMillan

Franklin R. McMillan, an ACI Honorary Member and active in Institute affairs since 1916, was honored recently by the University of Minnesota when the university conferred upon him the Outstanding Achievement Award, a special recognition to outstanding alumni of the university.

The award was established to recognize University of Minnesota graduates "who have brought distinction and credit to the University and to the professions in which they have carved new frontiers and established new bench-marks of growth. . . the presentation of these awards is, among other things, an expression of the deep obli-

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gations which all of us have for those who have lived courageously and fruitfully to the end that society in general benefits."

Mr. McMillan, consulting engineer, Asheville, N. C., was formerly manager of the Structural and Technical Bureau and Director of Research, Portland Cement Assn., Chicago.

Past president of ACI, Mr. McMillan was Wason Medalist for the most meritorious paper for his 1928 paper "Concrete Primer," which was released as a separate booklet by ACI with over 75,000 copies distributed, and authorized to be reprinted in many languages.

He is also the author of many JOURNAL contributions and has been active on numerous ACI technical and administrative committees.

Straub attends international conference

Lorenz G. Straub, director of the St. Anthony Falls Hydraulic Laboratory and head of the civil engineering department at the University of Minnesota, recently attended a series of international engineering conferences in Paris at the invitation of the United Nations Educational, Scientific and Cultural Organization (UNESCO). The purpose of the conferences was to establish a Union of International Engineering Organizations and to draft a constitution for such a federation.

Withycombe and Waite named to state posts

Earl Withycombe and Charles E. Waite have been made assistant state highway engineers for California, with Mr. Waite in charge of personnel and public relations. He succeeds J. G. Standley, retired. Mr. Withycombe takes the place of Ridgway M. Gillis, advanced to deputy state highway engineer, succeeding Fred J. Grumm, retired. Mr. Withycombe graduated from Oregon State University in civil engineering in 1911; in 1924 he became assistant construction engineer and in 1931 staff highway engineer. He was promoted to construction engineer of the Division of Highways in 1931. Mr. Waite is a graduate of the University of Oklahoma in civil engineering. During his service with the state he has been a designer, assistant district maintenance engineer, district engineer and design engineer.

Graham elected president

John W. Graham, Jr., Assistant to the Dean of the College of Engineering and Science and Assistant Professor of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa., has been elected president of the Pittsburgh chapter of the Pennsylvania Society of Professional Engineers.

Before joining the Carnegie faculty in 1946, Dr. Graham served as an engineer with the Fabricated Steel Construction Division of the Bethlehem Steel Corp.

In addition to the Society for Professional Engineers, he is also a member of ACI, ASCE and the American Society for Engineering Education.

Haselwood leaves highway department

Fred W. Haselwood has retired from the Department of Public Works, Division of Highways, of California after 38½ years, of which 26 were spent as district engineer in the three northern districts of the state. He will be a consulting engineer with headquarters at Redding, Calif.

Marquette plans new cement plant

Marquette Cement Manufacturing Company's board of directors recently approved the immediate construction of a multimillion dollar cement producing plant at Brandon, Miss. The new plant, which is expected to be completed about Sept. 1, 1951, will produce a yearly total of one million barrels of all types of portland cement.

Company officials point out that these facilities will greatly relieve the current scarcity of cement in the Mississippi market and in the portions of Louisiana and Arkansas also to be served by the new plant. At the present time there is no cement plant in the state of Mississippi.

Korean action emphasizes construction responsibilities

"It is our duty to study the problems which face the nation and the construction industry. and make recommendations on how the defense contsruction program, and all other construction which is vital to the growth and development of the nation, can be carried out now most rapidly and economically," declared Walter L. Couse, Detroit, Mich.,

before a recent meeting of the Associated General Contractors of America.

Mr. Couse, AGC president, reviewed the record of the construction industry during World War II and actions taken recently by AGC in the field of national defense.

During this period when the nation is resisting armed aggression in Korea, and preparing to resist it wherever it may break out in the world, the general contracting industry should take its responsibilities seriously, Mr. Couse stated.

Glenway W. Maxon, Dayton, Ohio, 1950 vice-president of the organization was nominated for president in 1951 to succeed Mr. Couse. Mr. Maxon is president of Maxon Construction Co., Inc., which specializes in heavy construction work.

International Congress of Prestressed Concrete

An International Congress of Prestressed Concrete will be sponsored by the Assn. of Engineers of Ghent during the celebration of its 75th anniversary in Ghent, Belgium, Sept. 10-13, 1951. The conference will pay special tribute to Eugene Freyssinet for his pioneering in the prestressed field and will present an up-to-date picture of theoretical and practical knowledge in this method of construction. The languages of the conference will be French, English and Dutch.

President of the organizing committee is Prof. Gustave Magnel, University of Ghent, and T. G. Riessauw, Chief Assistant, Laboratory for Prestressed Concrete, University of Ghent, is secretary.

New concrete almost like rubber

University of Minnesota researchers, in cooperation with the St. Paul Union Stockyards Co., are working to perfect a new type of concrete that will be fireproof, as strong as wood, and almost as resilient as rubber. Joseph Wise, technology professor, stated that the compressed concrete has stood ten times the strain needed to smash ordinary concrete planks, and that a concrete plank 1 in. thick and 8 ft long was deflected 12 in. before it broke, whereas an ordinary concrete plank would break after a ½-in. deflection.

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New Members

The 54 new Members added to ACI's membership list during October are divided into the following classifications—11 public service, 10 manufacturers of concrete products, 13 architects, engineers and contractors, 4 education, 1 PCA, 1 utilities and 14 students.

The membership total became 5203 on Nov. 1, 1950.

California

Atwater, John T., (Indiv.) c/o W. J. Burke & Co., 2257 E. 15th St., Los Angeles 21, Calif.
Halldin, Harold A., (Jr.) 1000 S. Marengo Ave., Alhambra, Calif.
Jakobsen, B. F., (Indiv.) 1325 Johnson St., Menlo Park, Calif.
Johnson, Arthur S., (Indiv.) 2765 Brant St., San Diego, Calif.
Johnson, Horace Allison, (Indiv.) 3640 Meadow Lane, Sacramento 19, Calif.
Leasure, Harry S. Jr., (Indiv.) 441 44th Circle, Long Beach, Calif.
Mangotich, Edward, (Jr.) 618 22nd St., Sacramento, Calif.
Sparolini, John A., (Indiv.) 4133 Marwick St., Long Beach, Calif.
Stalwick, R. W., (Indiv.) 1442 Harvard Ave., Fresno, Calif.
Valley Concrete Pipe & Products Co., Inc., (Corp.) P. O. Box 510, Chico, Calif. (J. J. Seale)
Witt, H. E., (Indiv.) Rt. 3, Box 1029, Escondido, Calif.

Colorado

Nordby, Gene M., (Jr.) Dept. of Civil Engr., University of Colorado, Boulder, Colo.

Connecticut

Mortenson, Ernest D., (Indiv.) 394 Hilltop Drive, Stratford, Conn.

Florida

deFelice, Frank W., (Indiv.) c/o Burnup & Sims, 505 Park St., W. Palm Beach, Fla.

Illinois

Dalal, Ramesh Chandra L., (Jr.) c/o DeLeuw Cather & Co., 150 N. Wacker Drive, Chicago 6, Ill.
Davis, Hermon S., Jr., (St.) 306 E. Hill, Champaign, Ill.
Desmond, Paul F., (Indiv.) 332 S. Cuyler Ave., Oak Park, Ill.
Gungen, Gunduz S., (St.) 906 S. Orchard St., Urbana, Ill.
Hernandez, Edmund Pena, (St.) 303 Thrush Ave., Peoria, Ill.
Janney, Jack R., (Indiv.) 9013 Lamon, Skokie, Ill.
Schoenrock, David H., (St.) 606 E. Chalmers St., Champaign, Ill.
Sigfusson, Benedict, (St.) 1111 S. First St., Champaign, Ill.

Iowa

Capper, Mack R., (Jr.) Bloomfield, Ia.

Kansas

Leonard, Chester L., (St.) 700 Saline, Topeka, Kans.

Kentucky

Curry, Milton B., (St.) 210 E. Second St., Lexington 42, Ky.

Maryland

Diserens, Robert L., (Indiv.) 8214 14th Ave., Hyattsville, Md.

Massachusetts

Hooper, William R., (Jr.) 27 Harriet Ave., Belmont, Mass.

Missouri

Porter, Charles Clifford III, (St.) Room 93, Civil Engineering Dept., University of Missouri, Columbia, Mo.

Nebraska

Barber, Ivan W., (Indiv.) Geology, Soils & Materials Section, Corps of Engineers, Missouri River Division, Omaha, Nebr.

New York

Hughes, H. Walter, (Indiv.) City Hall Annex, 54 Court St., Rochester 14, N. Y.
Lee, Chen-Hsi, (Indiv.) Apt. 2-D, 147-12 78 Road, Flushing, N. Y.

Ohio

Allen, Charles W., (Indiv.) Research Engineer, Ohio Highway Dept., 101 N. High St., Columbus 15, Ohio
Guran, John, (Indiv.) 1075 River St., Akron 5, Ohio

Oregon

Penepacker, Frank, (Indiv.) 400 N. Thompson St., Portland 12, Ore.
Steel, David F., (Indiv.) 2234 NE 79th Ave., Portland 13, Ore.
Wright, D. D., (Indiv.) 637 SE Water Ave., Portland, Ore.

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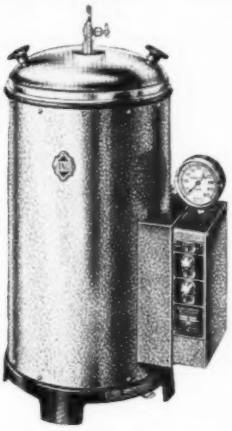

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Lendved, John W., (Indiv.) 7326 W. Cleveland Ave., Milwaukee 14, Wis.

Wyoming

Wilson, Lyle Lawrence, (Jr.) Recluse, Wyo.

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Aparicio, Rafael H., (Jr.) Box 22, Rio Piedras, P. R.
Costa, Arturo, (Jr.) Box 28, Rio Piedras, P. R.
Mendez, Rafael M. Jr., (Indiv.) P. O. Box 3586, San Juan, P. R.

Canada

Greenfield, William (St.), 393 Machray Ave., Winnipeg, Manitoba, Canada

England

Fraenkel, P. M., (Indiv.) 125, Victoria St., London S. W. 1, England
Schofield, R. W., (Indiv.) 16 Alfriston Rd., London S. W. 11, England

Guatemala

Carazo O., Noel R., (St.) 3a Avenida Sur No. 122, Guatemala City, Guatemala
Obiols G., Roberto Antonio, (St.) 6a Calle Oriente Jocotenango No. 4, Guatemala, Guatemala
Olivero V., Gmo. Rafael, (St.) 4 Calle Poniente No. 96, Guatemala, Guatemala
Urrudia F., Carlos Alberto, (St.) la Calle de Tivoli No. 38, Guatemala, Guatemala

Venezuela

Sous, Marcel V., (Indiv.) 2da Avenida Los Palos Grandes, Quinta Pin-Su- Chacao Edo. Miranda, Venezuela, S. A.

An engineer defines an engineer

*(and with notable clarity)**

"An engineer is a person who passes as an exacting expert on the basis of being able to turn out with prolific fortitude infinite strings of incomprehensible formulae calculated with micromatic precision from vague assumptions which are based on debatable figures taken from inconclusive experiments carried out with instruments of problematical accuracy by persons of doubtful reliability and questionable mentality for the avowed purpose of annoying and confounding a hopeless chimerical group of fanatics referred to, all too frequently, as engineers."

**Courtesy of* Chatter, *Engineering Society of Pennsylvania*

Honor Roll

February 1 to October 31, 1950

There are 340 Members listed on the Honor Roll October 31, 1950 representing 6½ percent of ACI's total membership. With three months to increase that percentage, why don't **you** join these Members and participate actively in sponsoring new Members?

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Chester P. Siess (Ill.)	6½
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Henry L. Kennedy (Mass.)	4
Cesar Oliver-Rugeles (Venezuela)	4
Ivan M. Viest (Ill.)	4
Wm. T. F. Hooper, Jr. (Ill.)	3½
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M. D. Hicklin (Ore.)	3
Samuel Hobbs (Calif.)	3
J. E. Jellick (Calif.)	3
James A. McCarthy (Ind.)	3
Stanley Moore (Canada)	3
Byram W. Steele (D. C.)	3
James L. Atkinson (Calif.)	2½
F. Thomas Collins (Calif.)	2½
H. J. Gilkey (Ia.)	2½
Castor Segundo Goa (Venezuela)	2½
Orlando W. Irwin (Ill.)	2½
W. H. Jacobs (Ill.)	2½
S. J. Warberg (Mich.)	2½
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Sterling Lowe Bugg (Fla.)	2
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J. J. Creskoff (D. C.)	2
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47th Annual Convention

February 20-22, 1951

HOTEL RESERVATIONS

A housing bureau has been organized for the 47th annual convention of the American Concrete Institute in San Francisco. Since all requests for rooms are handled in chronological order, it is recommended that you send in your application as quickly as possible.

Requests for hotel reservations, indicating first, second and third choice hotels should be addressed: **ACI Housing Bureau, c/o San Francisco Convention and Tourist Bureau, Room 200, 61 Grove St., San Francisco, Calif.** Because of the limited number of single rooms available, you will have a better chance of securing accommodations in a hotel of your choice if your request calls for rooms to be occupied by two or more persons.

ALL REQUESTS MUST BE ACCOMPANIED BY A DEPOSIT CHECK of $5.00 per person (or a minimum of $10.00 per room), made out to the **ACI Housing Bureau.** Due to the existing crowded conditions, hotels cancel unclaimed reservations by 6:00 p.m. Therefore, a deposit is requested to insure your reservation being held on your arrival day—whatever the hour. It will be credited to your account. PLEASE **DO NOT** SEND CASH or POSTAL NOTES.

All reservations will be confirmed if request is received not later than February 7th.

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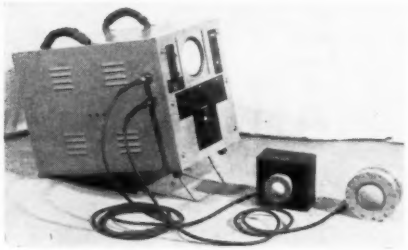
*Within one block of St. Francis Hotel.



Soniscope

An Instrument for indicating the quality of concrete on location.

Detects position of cracks in concrete—both in Field structures and Laboratory specimens. Uses Pulse Velocity techniques.

Reference—ACI Journal, Sept. 1949, Proceedings V. 46, pp. 17-36

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THE McPHAR ENGINEERING COMPANY of Canada, Limited

Cable: McPhar, Toronto **36 Cranfield Road, Toronto**

Licensed by: The Hydro-Electric Power Commission of Ontario

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Admixture produces stronger blocks

Denser, stronger and more uniform concrete blocks and bricks result from the addition to the mix of small amounts of a surface active agent, reports Monsanto Chemical Co.

The chemical, Santomerse S, a liquid wetting agent also employed in household and industrial cleaning compounds, is said to permit the use of less water by increasing its efficiency, thus producing a so-called "dry" concrete of greater strength. At the same time the plasticity required to properly mold blocks and bricks is improved. In addition, the wetting agent is said to result in cleaner equipment, lighter colored products and better dispersion of cement.

Further information is available from Monsanto Chemical Co., St. Louis 4, Mo.

Masonry drill bit

New England Carbide Tool Co. has developed a new carbide tipped masonry drill bit which is claimed to remove all dust as it drills. The manufacturer claims that this is made possible by two spiral low angle flutes turned in the steel drill body and by the fact that the diameter of the body is the same as the diameter of the hole to be drilled. The drills are available in sizes from $\frac{3}{16}$ to 1 in. diameter, up to 36 in. long. Further information is available from New England Carbide Tool Co., Cambridge 39, Mass.

Concrete saws

Three new portable concrete saws have been announced by Clipper Manufacturing Co. The saws have floating three-point suspension which, the manufacturer claims, eliminates blade binding as the saw moves over a rough surface. The cutting speed is 5 ft per minute, 1 in. deep, in limestone concrete. The maximum depth of cut is $6\frac{1}{2}$ in.

Many uses of concrete saws for patching, sawing utility trenches, setting poles, scoring designs and cutting contraction joints are illustrated in a leaflet available from Clipper Manufacturing Co., Concrete Saw Division, Dept. AC, 2800 Warwick, Kansas City 8, Mo.

Truck mixers

A 20-page bulletin released by Worthington Pump and Machinery Corp. illustrates and explains the various features of truck mixers and agitators. Specifications, and line drawings are included. Copies of Bulletin R-1700-B 3 can be obtained from Worthington Pump and Machinery Corp., Harrison, N. J.

Tilting mixer

Eliminating elevated foundations or separate mixer platforms, a tilting-type concrete mixer developed by Supremix, Inc., is an integral part of the plant structure. Due to its unique tilting arrangement, the manufacturer states, the mixer is located lower in the structure and at the same time allows sufficient height for delivery equipment. The mixer tilts hydraulically directly into the delivery equipment, thus eliminating the collecting hopper.

The method of blading used, claims the company, effects a rope action exchange in mixing the ingredients from the ends of the mixer, giving fast, thorough mixing and the mixer is easy to keep clean. A consistency meter is also attached.

Further information can be obtained from Supremix, Inc., Adrian, Mich.

Who's Who

Continued from p. 12

Board of Direction and has served on various technical and administrative committees, as well as author of several contributions to JOURNAL pages.

Associated with various cement companies ever since graduation from Case Institute of Technology, he was appointed Director of Research for Lone Star in 1934.

F. Tessitor and P. Rosewarne

F. Tessitor and P. Rosewarne, U. S. Bureau of Reclamation, Denver, Colo., discuss the possibilities of "Economy Through Better Control of Reinforcing Steel," p. 333.

Mr. Tessitor, Head of Metal Laboratory group of Research and Geology Division, joined the Bureau in 1937 and has been engaged in materials investigations and research continuously. Graduating from Washington State College in 1930, he was associated with Bell Telephone Laboratories and the Corps of Engineers prior to joining the Bureau. A portion of the Bureau work has been investigations of the causes and means of alleviating failures and sources of trouble of engineering materials in various projects.

Mr. Rosewarne, materials engineer, has been with the Bureau since 1946. He graduated from Michigan College of Mining and Technology in 1938 and was employed at the Gary Works of Carnegie Illinois Steel Corp. and Army Ordnance before seeing service with the U. S. Navy.

Washington Regional Meeting

Continued from p. 6

pressive strength of all mixes decreased about 100 psi for each one percent increase in air content.

Shrinkage was about the same as in normal concrete, saturation coefficient was much lower and the resistance to freezing and thawing was generally higher than for conventional dense concrete.

Inspection Trip

The climax of the three-day meeting was an inspection trip Thursday, October 26, to the site of the Chesapeake Bay bridge now under construction near Annapolis, Md. Two bus loads left the Mayflower about 9:30 and were met at the Gate of the U. S. Naval Academy about 10:30 by Capt. A. D. Hunter, Deputy Chief of the Bureau of Yards and Docks, and Capt. H. B. Jones, Public Works Officer of the Adacemy, through whose efforts a tour of the Academy buildings and grounds had been arranged. At noon the party left the Academy for lunch at famed Carvel Hall.

J. E. Dunn, chairman of the trip committee, introduced Hershall H. Allen, President, J. E. Greiner Co., consulting engineers for the bridge, who in turn introduced other special guests including: William F. Childs, Jr. and Walter Hopkins, Chief Engineer and Deputy Chief Engineer, respectively, Maryland State Road Commission; R. M. Reindollar and Russell M. McCain, Chairman and Member, respectively, Maryland State Road Commission; Harry M. Brown, Bruce Herman, Joseph F. Knonerle, W. H. Paul and Eugene Lloyd, all of J. E. Greiner Co.; and Richard Tower, J. Rich Steers, Inc.

After lunch the party again rode busses to the bridge site where two tugs and a launch carried it to various units under construction in the bay for a close-up of concreting operations on abutments, open cofferdam-type piers, Potomac-type piers and main anchorages.

Busses returned the party to the Mayflower about 5:30.

Papers to be published

All papers presented at the Washington meeting are being considered for JOURNAL publication and with the cooperation of the authors will be published as soon as possible.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 900-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.

The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.

The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.

A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.

The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.

The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly anchorages; uses large diameter wires (⅝-in. diameter now available on the market), instead of the customary ¼-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

For more than 30 years the design and building of concrete and reinforced concrete structures have been guided by the work of Joint Committees.

Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete

This book (published 1940) represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association.

Recommended Practice
- Materials
- Proportioning, mixing, curing and testing
- Forms and placing
- Waterproofing and protective treatments
- Surface finishes
- Design

Standard Specifications
- Materials
- Proportioning
- Measurement
- Mixing
- Curing
- Field tests
- Forms and placing
- Details of construction

$1.50 (to ACI Members—$1.00)

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Proposed Revision of BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-47).....47-18

Price 35 cents.

REPORT OF COMMITTEE 318—Dec. 1949, pp. 269-276 (V. 47)

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special achorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

CURING CONCRETE PAVEMENTS WITH MEMBRANES.....47-19

Price 35 cents.

C. C. RHODES—Dec. 1949, pp. 277-296 (V. 47)

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensive as wet curing under the same conditions.

BLADE CHANGES IMPROVE TILTING MIXER.....47-20

Price 35 cents.

GLENWAY MAXON—Dec. 1949, pp. 297-300 (V. 47)

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

PROTOTYPE PRESTRESSED BEAM JUSTIFIES WALNUT LANE BRIDGE DESIGN.....47-21

Price 35 cents.

GUSTAVE MAGNEL—Dec. 1949, pp. 301-316 (V. 47)

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

FINISHING AND CURING: A KEY TO DURABLE CONCRETE SURFACES.....47-22

Price 35 cents.

MYRON A. SWAYZE—Dec. 1949, pp. 317-332 (V. 47)

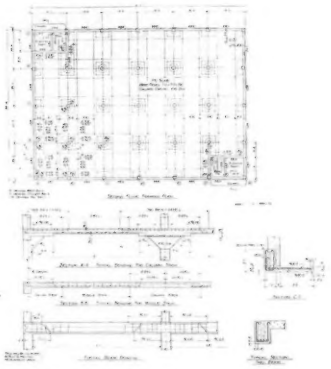
Continued on p. 30

Manual of Standard Practice for Detailing Reinforced Concrete Structures

(ACI Standard 315-48)

for the

Designer
Detailer and
Student



$3.00 (to ACI Members—$1.75)

The **Manual** presents accepted methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. The use of these methods and standards results in better reinforced concrete construction and also simplifies and reduces the amount of work ordinarily required to prepare such drawings.

The fundamental purpose of all "engineering drawings" is to enable the builder to transform the designer's calculations and design into the structure he has conceived. These drawings give the information necessary for building forms and placing concrete and for preparing the "placing drawings" (sometimes called "detail drawings") which must contain the information necessary for fabricating the reinforcement and placing it in the forms.

The **Manual** illustrates typical engineering and placing drawings prepared according to the standards. The designs are not intended as standard designs, although in general they meet the requirements of the American Concrete Institute's "Building Code Requirements for Reinforced Concrete" (ACI Code). They are intended to show **standard detailing methods.**

American Concrete Institute 18263 W. McNichols Rd. Detroit 19, Mich.

The Detailing Manual includes—

Drawing standards—Standards and comments on size of sheet, layout, scales and symbols.

Marks—Discussion of systems of marking members and reinforcement.

Schedules—Discussion of types of schedules and recommendations for their use.

Engineering drawings—Information required on drawings and best methods of showing it.

Placing drawings—Best methods of showing necessary information.

Fabricating detail standards—Standards for bending, hooks, stirrups, typical bar bends, slant lengths, beam widths, spirals, ties, column verticals, splices and dowels.

Notes to designers and detailers—Attention is called to points which frequently are overlooked or which cause trouble. Conformance with the suggestions given eliminate most difficulties of interpretation, fabrication and construction.

Shop procedure—Brief description of steps in the shop.

Warehouse stock—Notes material usually carried in stock.

Tolerances—Standard tolerances in dimensions of fabricated reinforcement.

Extras—Basis for extra charges for material and fabrication.

Welded wire fabric—Data on fabrication, size limitations, detailing and common styles.

Accessories—Standard accessories and their use.

Typical drawings—21 typical engineering and placing drawings are shown for various structures to illustrate the use of the standards and methods advocated in the manual. A short discussion of the important points accompanies each drawing.

Typical drawings

The engineering drawings used in the manual were selected from plans of actual structures but have been modified in some details to conform more closely with the ACI Code and to illustrate recommended methods of presenting the information. Shop prepared placing drawings accompany each engineering drawing.

- Concrete joist floor
- Flat slab floor
- Beam and girder floor
- Two-way slab and beam floor
- Foundations
- Columns
- Architectural concrete building
- Wall details
- Stair details
- Circular tank
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Synopses

Continued from p. 27

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

ECONOMY THROUGH BETTER CONTROL OF REINFORCING STEEL.....47-23

Price 35 cents.

F. TESSITOR and P. ROSEWARNE—Dec. 1949, pp. 333-340 (V. 47)

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel would, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

Changes in the "Detailing Manual" are proposed to conform to changes in the Building Code (December Journal) allowing new bond values for new-style deformed reinforcing bars.

Proposed Revisions to

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)*

Reported by ACI Committee 315

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SYNOPSIS

Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

The adoption of the new-style deformed reinforcing bars and proposed changes in bond values† have made it mandatory that "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)" be brought into conformity with these new developments.

Changes and revisions proposed fall into four general catagories:

1. Revisions brought about by the new-style deformed bars

All references to bars shall be changed from diameter or side measurement and shape to numbered designations in accordance with U. S. Department of Commerce Simplified Practice Recommendation R26–50 and Table 1.

*Title No. 47-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 5, Jan. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than May 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

This report was approved in form and substance as here submitted by the following members of the committee: Raymond Archibald, Frank H. Beinhauer, Everett E. Ebling, Frank Kerekes, Henry L. Neve, Fred L. Plummer, Raymond C. Reese, E. E. Rippstein and John F. Seifried. It is released by the Standards Committee for publication and discussion with view to its consideration for adoption at the 47th annual convention, San Francisco, Calif., Feb. 20-22, 1951.

†See "Proposed Revision of Building Code Requirements for Reinforced Concrete (ACI 318-47)" reported by Committee 318, ACI JOURNAL, Dec. 1950, *Proc.* V. 47, p. 269.

TABLE 1—STEEL REINFORCING BAR DESIGNATIONS AND PROPERTIES*

Obsolete bar designation, size	Current bar designation, No.†	Unit wt per ft, lb	Nominal dimensions—Round sections		
			Diameter, in.	Cross sectional area, sq in.	Perimeter, in.
¼-in. ϕ	# 2‡	0.167	0.250	0.05	0.786
⅜-in. ϕ	# 3	0.376	0.375	0.11	1.178
½-in. ϕ	# 4	0.668	0.500	0.20	1.571
⅝-in. ϕ	# 5	1.043	0.625	0.31	1.963
¾-in. ϕ	# 6	1.502	0.750	0.44	2.356
⅞-in. ϕ	# 7	2.044	0.875	0.60	2.749
1 -in. ϕ	# 8	2.670	1.000	0.79	3.142
1 -in. □	# 9§	3.400	1.128	1.00	3.544
1⅛-in. □	# 10§	4.303	1.270	1.27	3.990
1¼-in. □	# 11§	5.313	1.410	1.56	4.430

*U. S. Dept. of Commerce Simplified Practice Recommendation R26-50.
†Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.
‡Bar # 2 in plain rounds only.
§Bars # 9, # 10 and # 11 correspond to former 1-in. square, 1⅛-in. square and 1¼-in. square sizes and are equivalent to those former standard bar sizes in weight and nominal cross sectional areas.

Example—References such as: 3—½″ ϕ @ 6″ shall be changed to 3—# 4 @ 6″

Appropriately all drawings, tables of individual bars, tables relating to laps, spacings, etc., shall be corrected to show bar numbers instead of nominal diameters.

2. Changes brought about by the higher bond values recommended for adoption in "Proposed Revision of Building Code Requirements for Reinforced Concrete (ACI 318-47)"

Appropriately on all drawings where the "revised Code" permits, hooks and special anchorages shall be omitted, laps and splices reduced, and lengths of bars and bending details changed (Fig. 1).

3. Changes to make the Manual conform to the ACI Code

In the few instances of nonconformity, the "Detailing Manual" shall be revised to conform to the ACI Code.

4. Textual changes

Changes of an editorial nature are required to improve cross references, to correct minor misstatements and typographical errors, to clarify explanations, and similar small modifications.

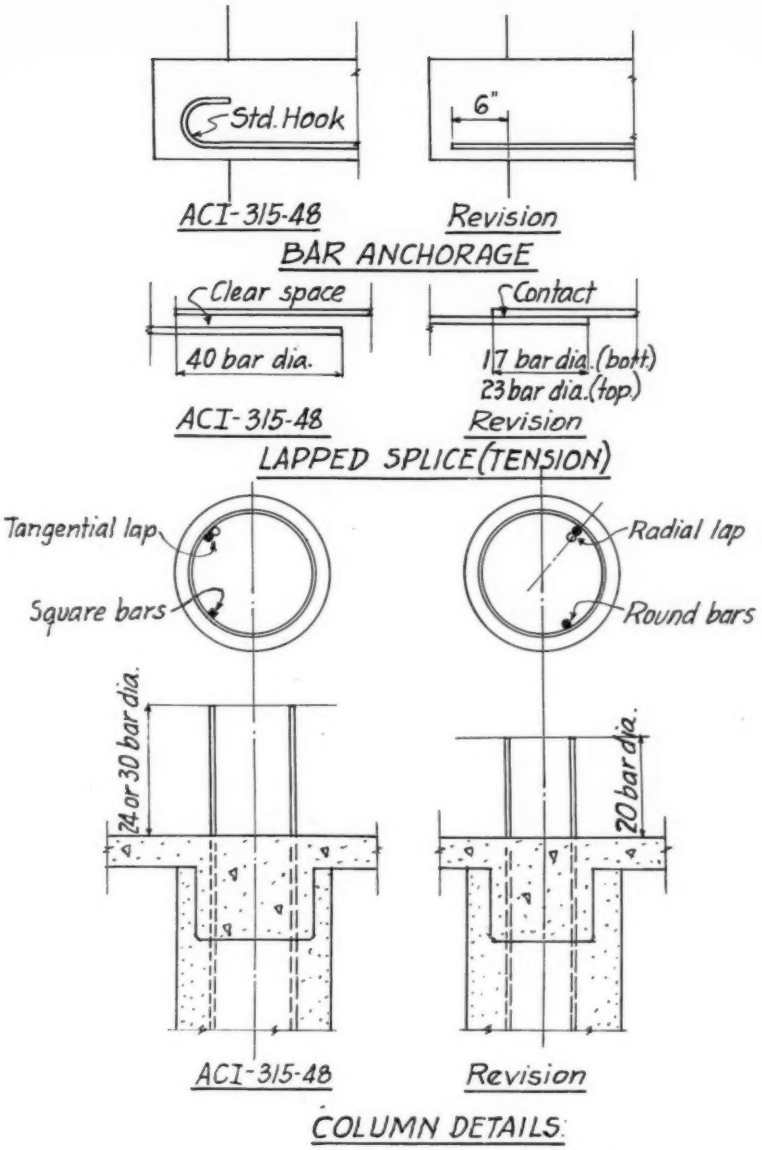

Fig. 1—Examples of changes in details to incorporate new Building Code provisions for new-style deformed reinforcing bars

Title No. 47-25

In recent Journal papers Jackson and Swayze discussed the effect of construction methods, aggregate grading, finishing and curing on the durability of concrete surfaces. Here cement is examined and its degree of fineness assessed as a factor in concrete durability.

Coarse-Ground Cement Makes More Durable Concrete*

By HAROLD W. BREWER and RICHARD W. BURROWS†

SYNOPSIS

Test procedures for measuring the cracking resistance of neat cement, mortar and concrete are described. A relationship between shrinkage cracking and resistance to freezing and thawing is shown. These laboratory tests indicate that coarse-ground cement produces more durable concrete than fine-ground cement.

INTRODUCTION

In recent years many engineers and contractors have preferred fine-ground cements because they improve the workability of the fresh concrete and the early strength development of the hardened concrete. There are strong indications, however, that such fine-ground cements may not be as weather resistant as the coarser cements used many years ago. Jackson mentions this in "The Durability of Concrete in Service,"[1] while others have expressed the same belief. Carlson, in his paper "Remarks on the Durability of Concrete,"[2] suggested that coarse cement produces concrete with a greater ability to resist cracking; and that coarse cement provides a reserve of unhydrated cement for the possible healing of fine cracks in the concrete.

It has been observed in laboratory tests that concretes made with fine-ground cement were less resistant to accelerated freezing and thawing than those made with coarse-ground cement when they were dried after preliminary moist curing. Microscopic examination of these specimens showed that the concretes made with the finer cement contained more fine cracks than the ones made with the coarser cement. A test program was therefore planned to investigate concretes made with cements of different fineness. Specimens were made with three cements each covering a range of fineness with specific

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†Members American Concrete Institute, Materials Engineer and Engineer, respectively, U. S. Bureau of Reclamation, Denver, Colo.

surfaces from 1200 to 2700 sq cm per g (Wagner). The specimens were cured under different conditions because it had been demonstrated previously that the length of moist curing and drying had a material effect on the resistance of the concrete to freezing and thawing. Restrained mortar specimens and neat cement slabs were also made to demonstrate the cracking tendencies of the cements of different finenesses.

The tests described show that concrete made with the coarser cement and subjected to a period of weathering before starting the laboratory freezing-and-thawing cycles is more resistant to cracking and to freezing and thawing than concrete made with fine-ground cement.

TEST PROCEDURE

Three cement clinkers, A, B and C, having the chemical compositions shown in Table 1 were ground in a laboratory ball mill to finenesses varying from 1200 to 2700 sq cm per g in increments of 300 sq cm per g.

The cracking tendencies of the cements were tested by casting them in rings of mortar around rigid metal discs and observing the time required for the rings to crack from drying shrinkage, in accordance with Carlson's tests.[3] (Fig. 1). The mortars were composed of 2.34 parts of sand and had water-cement ratios of approximately 0.46.

TABLE 1—CHEMICAL ANALYSIS AND COMPOUND COMPOSITION OF CEMENTS

Chemical Analysis

Cement	A	B	C
Type	I	II	I
SiO_2	20.84	25.55	21.60
Al_2O_3	6.19	3.01	5.95
Fe_2O_3	2.71	1.70	2.77
CaO	65.09	64.95	63.32
MgO	0.55	1.49	1.33
SO_3	2.05	1.15	1.86
Loss	1.47	1.46	2.19
Na_2O	0.22	0.18	0.11
K_2O	0.70	0.53	0.49
Mn_3O_4	0.02	0.09	0.21
Insoluble	0.20	0.12	0.27
$CaSO_4$	3.5	1.96	3.2

Compound Composition

C_3S	55.1	44.0	44.2
C_2S	18.2	40.1	28.6
C_3A	11.8	5.1	11.1
C_4AF	8.2	5.2	8.4

Drying shrinkage was measured on unrestrained mortar bars cured and dried under the same conditions as the restrained mortar rings. The cracking tendency of cements of different fineness was also compared by fabricating neat cement slabs internally restrained with No. 4 wire screen. These slabs (Fig. 2) were fog cured for 14 days and then dried at 70 F and 50 percent relative humidity. A mixture of one part of cement to 0.5 part of water was used in all neat cement slabs.

Aggregate graded up to ¾-in. maximum size was used in the 3 x 6-in. concrete cylinders for the freezing-and-thawing tests. The proportions of cement to aggregate and the slumps were held constant in each series. The water content varied slightly with the fineness of the cement. The mix proportions and concrete properties are shown in Table 2.

Before starting the freezing-and-thawing tests the concretes were cured by one of the following methods: (1) continuously fog cured and then immersed in water for 24 hours, (2) fog cured for 14 days, followed by drying at 70 F and 50 percent relative humidity, and then immersed in water for 72 hours, or (3) fog cured for 14 days, followed by outdoor storage for at least 1 year before immersion in water for 72 hours.

After curing, the concretes were alternately frozen at 10 F for 1½ hours and thawed at 70 F for 1½ hours while immersed in water. Weight and length measurements were made at frequent intervals during the freezing-and-thawing tests until the specimens had lost 25 percent of their weight.

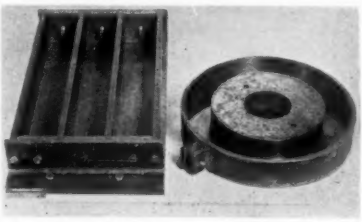

Fig. 1—Molds and disc used for ring shrinkage tests

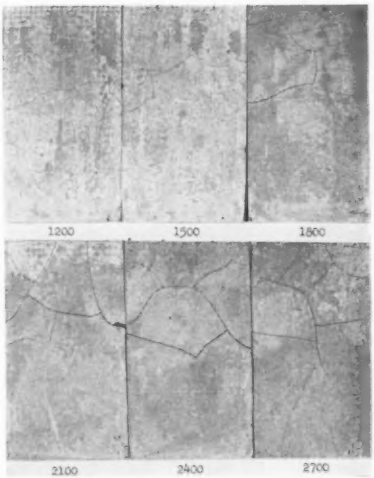

Fig. 2—Effect of cement fineness (Wagner sq cm per g) on shrinkage cracking of restrained neat cement slabs—one-eighth actual size

TEST RESULTS AND DISCUSSION

When a ring of mortar was restrained by casting around a smooth metal disc as described previously, it was found, as shown in Fig. 3, that the mortars containing the coarser cements resisted cracking for the longest drying period. Shrinkage measured on unrestrained companion bars showed that mortars containing the coarser cements shrank at a slower rate but had as much as 40 percent more drying shrinkage at the time the restrained mortar ring cracked, indicating that mortars made with coarse cement are much more resistant to cracking from drying shrinkage than those made with finer cements.

The greater resistance of coarse ground cement to cracking resulting from drying shrinkage was shown qualitatively by drying internally restrained slabs of neat cement. Fig. 2 shows plainly that the slabs made with the coarser cements contained fewer cracks. Unrestrained specimens of larger cross section have been found to crack in a similar manner.

In mixing the concrete for the freezing-and-thawing specimens with cements A and B, the water required to produce a constant slump decreased

TABLE 2—MIX DATA AND TEST

Cement and aggregate	Cement fineness, sq cm per g (Wagner)	Mix proportions by weight				Slump, in.	Unit weight, lb per cu ft
		W	C	S	G		
Cement A (Type I), Friant Dam aggregate	1200	0.52	1.00	2.30	3.40	5	147.7
	1500	.51	1.00	2.30	3.40	6	148.2
	1800	.49	1.00	2.30	3.40	5	148.6
	2100	.49	1.00	2.30	3.40	5 1/2	148.8
	2400	.51	1.00	2.30	3.40	5 1/2	148.5
	2700	.54	1.00	2.30	3.40	5 1/2	148.7
Cement B (Type II), Platte River aggregate	1200	.53	1.00	2.24	2.24	2 5/8	144.4
	1500	.53	1.00	2.24	2.24	3 3/8	144.1
	1800	.51	1.00	2.24	2.24	3 1/8	144.5
	2100	.50	1.00	2.24	2.24	2 7/8	145.6
	2400	.51	1.00	2.24	2.24	3 1/8	143.5
	2700	.51	1.00	2.24	2.24	3 1/8	145.5
Cement C (Type I), Platte River aggregate	1200	.50	1.00	2.15	2.15	2 3/4	143.9
	1500	.49	1.00	2.15	2.15	2 3/4	144.9
	1800	.48	1.00	2.15	2.15	2 1/4	145.3
	2100	.48	1.00	2.15	2.15	2 1/4	145.3
	2400	.46	1.00	2.15	2.15	2 1/8	145.6
	2700	.45	1.00	2.15	2.15	3	145.9

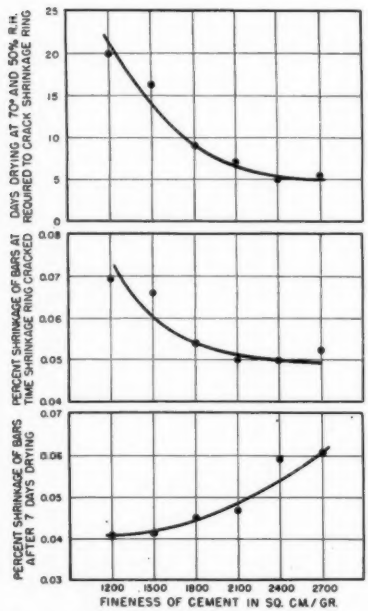
DAYS DRYING AT 70° AND 50% R.H. REQUIRED TO CRACK SHRINKAGE RING
PERCENT SHRINKAGE OF BARS AT TIME SHRINKAGE RING CRACKED
PERCENT SHRINKAGE OF BARS AFTER 7 DAYS DRYING
1200 1500 1800 2100 2400 2700
FINENESS OF CEMENT IN SQ. CM./GR.

Fig. 3—Coarse-ground cement delayed cracking of shrinkage ring.

as the fineness of the cement was increased up to about 2100 sq cm per g where the water requirement began to increase with further increase in fineness. With cement C, the water required to produce a constant slump was reduced as the cement fineness was increased for the full range of fineness tested. Fig. 4 shows that when the concrete specimens were kept continuously moist before subjecting them to cycles of freezing and thawing, the resistance of the concrete was related to the water content—the lower the water content, the higher the resistance.

When the concretes were dried, however, durability was no longer related to water content. Fig. 5 compares the continuously fog-cured data with that obtained on companion concretes fog-cured for 14 days, but dried at 50 percent relative humidity before freezing and thawing. For cement C the durability curve was completely reversed by drying. Cement B showed an improvement for the coarse cement and a retrogression for the finer cements. The same trend is indicated for cement A, but probably a lower

RESULTS FOR CONCRETE SPECIMENS

Water, lb per cu yd	Cement, lb per cu yd	Air, percent	Shrinkage, percent	Drying weight loss, percent	Absorption, percent	Cycles of freezing and thawing		
						Fog cured	Fog cured 14 days and dried	Fog cured 14 days and exposed
292	560	2.2	0.047	3.1	3.2	170	128	178
288	564	2.1	.048	2.7	2.9	158	140	178
277	568	2.2	.049	2.4	2.6	300	200	172
278	568	2.1	.047	2.2	2.6	245	156	110
288	564	1.9	.048	2.1	2.5	218	148	90
304	564	1.1	.049	2.3	2.9	150	90	60
341	646	1.9	.061	4.1	4.3	50	85	200
341	645	2.0	.071	4.0	4.1	100	100	200
331	649	2.2	.070	3.6	3.8	190	125	175
328	655	1.6	.071	3.3	3.4	240	90	130
329	644	2.8	.069	3.2	3.3	210	90	100
333	653	1.5	.071	2.9	3.2	190	85	80
336	666	2.0	.062	4.1	4.1	90	280	170
330	672	1.6	.069	3.6	3.8	140	280	155
326	675	1.5	.065	3.3	3.4	250	225	100
326	675	1.5	.069	3.2	3.3	350	170	93
316	679	1.7	.067	2.9	3.3	350	130	80
309	682	1.7	.070	3.1	3.2	500	125	50

fineness would be required to show an improvement for the coarse cement when the specimens are dried prior to testing. The effect of drying shown in Fig. 5 is believed due to the more severe microscopic shrinkage cracking which occurs with the fine-ground cement, as shown in the photomicrographs in Fig. 6.

Fig. 7 shows that when the specimens were exposed outdoors for 1 year or more, there was a pronounced drop in the resistance of the concretes to freez-

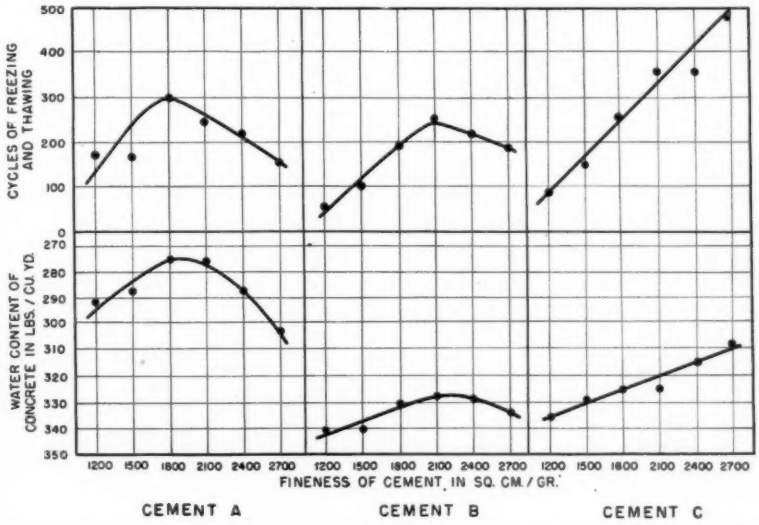

Fig. 4—Tests on continuously fog-cured specimens. Other factors being equal, resistance to freezing and thawing depended on water content.

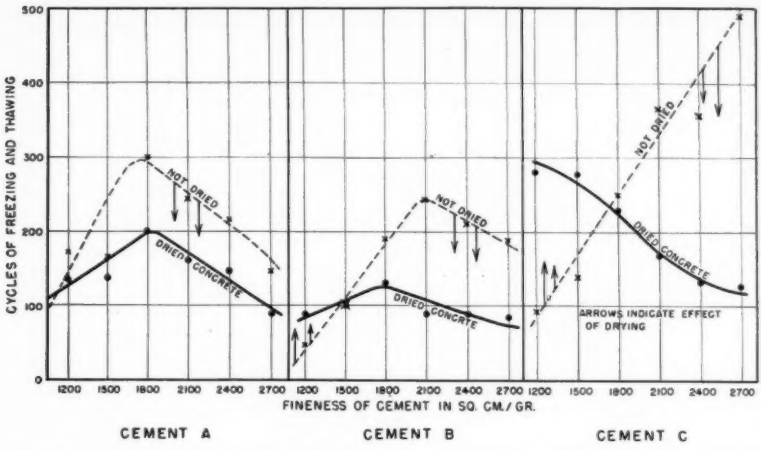

Fig. 5—Tests on dried concretes. Drying reduced freezing-and-thawing resistance of concretes containing fine-ground cements.

ing and thawing as the fineness of the cements increased. The outside exposure was in the semi-arid climate near Denver, Colorado, and wetting and drying probably caused more cracking than the single drying of the previously discussed series. The test results for the specimens subjected to outside exposure are believed to be of greater significance than those obtained on continuously moist-cured specimens or on the specimens dried once at constant temperature and humidity.

An interesting difference between coarse and fine cements has been observed by dye patterns on sectioned concrete specimens. The sections were sawed

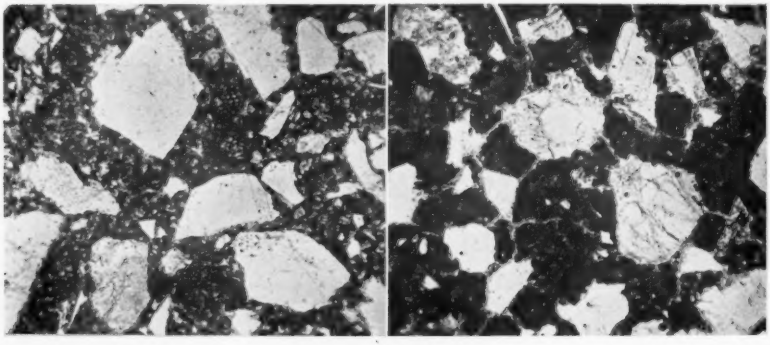

Fig. 6—Photomicrograph showing microscopic shrinkage cracks in concrete containing fine ground cement. Magnification: 42X. Fineness of cement (Wagner): Left—1200 sq cm per g. Right—2700 sq cm per g

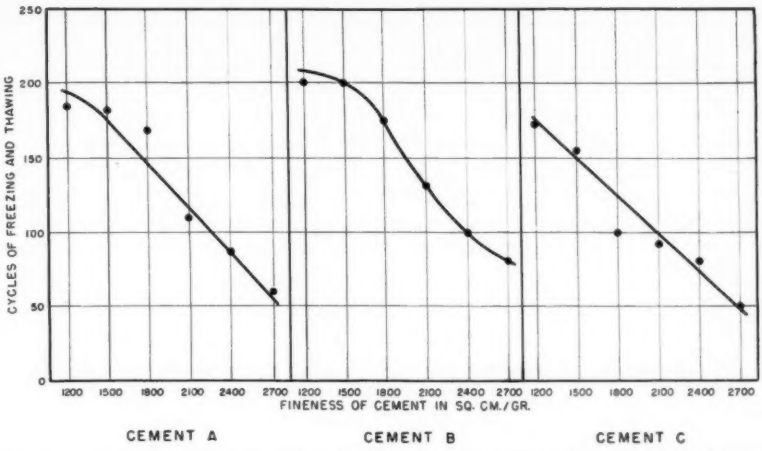

Fig. 7—Tests on weathered concretes. Coarse-ground cement produced more durable concrete.

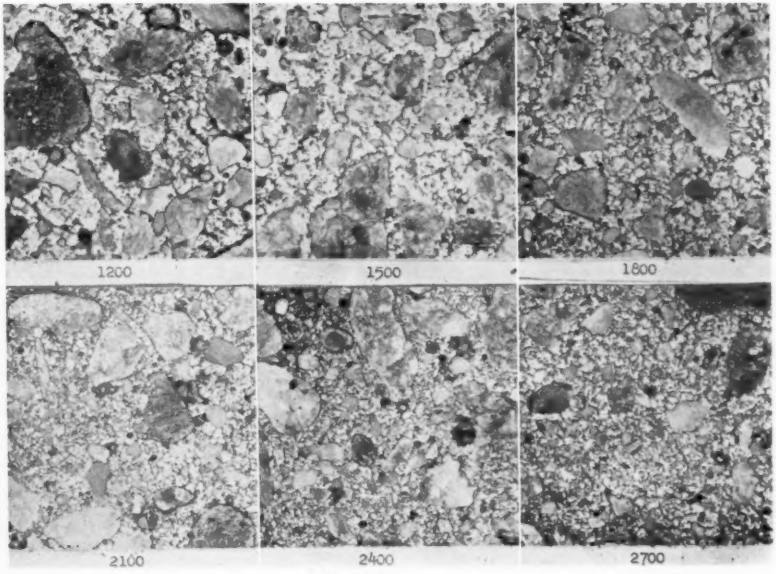

Fig. 8—Dye patterns obtained on dried concretes—actual size. Cement fineness (Wagner) in sq cm per g

from dry concrete specimens, immersed in dye solution, and the sawed surface ground until a pattern of stained and unstained areas became visible. In Fig. 8 the coarse aggregate boundaries in the coarse-cement concrete are heavily stained with very little stain in the mortar, while the mortar of the fine-cement concrete is heavily stained with little evidence of staining around the coarse aggregate. These patterns, indicative of the distribution of water in the concrete, may account for the typical scaling of the mortar from the coarse aggregate which results when a concrete containing a coarse cement is frozen and thawed. A concrete containing fine-ground cement usually manifests less surface scaling but a greater expansion, indicating that the disruption from freezing and thawing is occurring within the mortar, rather than at the coarse aggregate boundaries.

It can be concluded from these test results that concrete made with coarse-ground cement is more resistant to cracking resulting from drying, and that it has a greater resistance to freezing and thawing under normal exposure than concrete made with fine-ground cement.

ACKNOWLEDGMENTS

The authors wish to acknowledge the assistance of W. H. Price, H. S. Meissner and R. F. Blanks in the guidance of the tests and the review of this paper.

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Discussion of a paper by Harold W. Brewer and Richard W. Burrows:

Coarse-Ground Cement Makes More Durable Concrete*

By HUBERT WOODS, F. O. ANDEREGG, M. SPINDEL, BRYANT MATHER and AUTHORS

By HUBERT WOODS†

It would be unfortunate if the title and conclusion of this paper were taken to indicate that the way to frost resistant concrete lies in the use of coarse-ground cement. We do not believe that it does. It is well established that in concrete made with usual water-cement ratios, the hardened portland cement paste contains a surprisingly large internal capillary pore volume, capable of holding water that can freeze. If through long contact with water, this capillary pore space becomes saturated or nearly saturated with water, repeated freezing and thawing will damage the hardened paste at a rate depending upon several factors, unless the hardened paste has been properly protected with entrained air.[1]

This conclusion appears to be generally valid over the entire range of composition and fineness of commercial portland cement. It is supported by extensive fundamental and experimental research in the Research and Development Laboratories of the Portland Cement Assn.

In recent experimental work, concrete prisms without entrained air, made with four of the ASTM types of cement (insofar as composition is concerned) and generally covering the range of fineness from 1200 to 2500 sq cm per g, were found without exception to be vulnerable to freezing and thawing in the near-saturated state. All expanded 0.1 percent in about 100 cycles or less. On the other hand, some of these same concretes (in fact, some of those which had deteriorated very rapidly) were duplicated but with properly entrained air, and these have now exceeded 1500 cycles and still show very low expansion.

The function of entrained air is not only to provide available space to accommodate the increase in volume resulting from the conversion of water to ice, but also to limit the distance through which water, as yet unfrozen, must move through the paste as that water is displaced by ice. In air entrainment we seem to have a unique means of preventing the deterioration of hardened cement paste under the influence of freezing and thawing. This is not the same as saying that entrained air will always prevent the deterioration

*ACI JOURNAL, Jan. 1951, *Proc.* V. 47, p. 353. Disc. 47-25 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

†Director of Research, Portland Cement Assn., Chicago, Ill.

of *concrete* subjected to freezing and thawing. No such claim is made. All we can expect from entrained air, with respect to the resistance of concrete to freezing and thawing, is that it will protect the hardened cement paste. There are many other factors involved in the durability of *concrete*.

The point is that the authors of the present paper tested concrete specimens, all of which were inherently vulnerable to freezing and thawing in the saturated condition. Thus they were dealing with differences in rates of deterioration of various specimens, none of which could be classified as frost resistant, whatever the cement fineness. In the light of present knowledge, it does not seem too hazardous to conclude that frost resistant concrete cannot be expected to result from manipulation of cement fineness.

The authors use the number of cycles of freezing and thawing required to produce a weight loss of 25 percent as a criterion of performance. It is believed that this criterion is open to question. If the conditions of test are suitable, surface scaling and some weight loss may take place, even though the specimen retains most of its structural integrity, as shown by very low expansion and high modulus of elasticity. Conversely, under different test conditions, the concrete may be severely damaged structurally, as shown by excessive expansion and lowered modulus of elasticity, while undergoing little if any weight loss, or even gaining weight. It might be useful if the authors would add their expansion measurements to the record.

In the present paper, within each series of finenesses of the same cement composition, the SO_3 content of the cements appears to be the same. The work of Lerch[2] and of Pickett[3] suggests strongly that the optimum SO_3 content, with respect to shrinkage, depends upon cement fineness. To some extent, then, the larger shrinkage and cracking noted for the finer cements may indicate that these cements did not contain enough gypsum.

A very interesting feature of the present paper is the complete reversal of the observed "effect" of fineness, consequent upon a change in experimental procedure. One may well speculate whether differences in experimental procedures or test methods might account for some of the conflicting results and conclusions appearing not infrequently in the literature on concrete. Perhaps we should take more seriously the advice given some years ago by Arthur S. Eddington, in the following quotation:

> "But are we sure of our observational facts? Scientific men are rather fond of saying pontifically that one ought to be quite sure of one's observational facts before embarking on theory. Fortunately those who give this advice do not practice what they preach. Observation and theory get on best when they are mixed together, both helping one another in the pursuit of truth. It is a good rule not to put overmuch confidence in a theory until it has been confirmed by observation. I hope I shall not shock the experimental physicists too much if I add that it is also a good rule not to put overmuch confidence in the observational results that are put forward *until they have been confirmed by theory.*"

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By F. O. ANDEREGG*

In considering the durability of concrete, a variety of factors must be considered. The authors stressed shrinkage cracking and resistance to freezing and thawing, which involve different mechanisms.

The first is determined by (1) the amount of gel present; (2) by the distension of the gel, which is correlated with the amount of water used in the mix; (3) by the history of the gel which involves the partially reversible shrinkage and distension with alternate drying and wetting; and (4) the grading of the aggregate, included with which the larger cement particles must be incorporated. With a well graded system these particles come in close juxtaposition, and resist shrinkage forces by virtue of their low compressibility. It has been demonstrated that graded *cements*† exhibit less shrinkage than commercial cements from the same clinker, with a logarithmic grading being most effective. The last had greater specific surface but the better fit of the particles was best able to resist the shrinkage forces. Similarly, proper grading of aggregate particles will assist such resistance. It was pointed out that humps in the grading curve of the cements produced more or less serious interference with the packing and hence allowed more shrinkage to occur. It is respectfully suggested that the cements used by Brewer and Burrows contained a variety of hump sizes, especially in the fine micron region, where greatest harm can be done. It is also suggested that in air-separated cement more humps may be anticipated than in cement produced by continuous grinding.

The resistance to freezing depends upon the relation of the hydraulic pressure developed during the freezing and the cohesive strength of the concrete to resist it. Resistance to this pressure may be helped by the incorporation of air, as is well known. Now the well graded cement produces a dense structure which has higher strength, it is true; but which also is readily affected by hydraulic pressure. Hence in such systems subjected to freezing and thawing, buffer action, such as entrained air gives, should be provided.

Another factor in durability is the action of a variety of chemically-corrosive agents, such as acid and sulfate waters. Soluble salts, including especially the sulfates of sodium, magnesium and calcium, in addition to the chemical attack, are capable of slowly growing into crystals within the concrete and developing disruptive forces of the order of 100,000 psi. Both corrosive action and efflorescent crystal growth are reduced and resisted by a very dense paste structure, such as obtained with the properly graded cement.

*Director of Building Materials Research, John B. Pierce Foundation.

†Anderegg, F. O., "Particle Size Distribution of Portland Cement and What it Means," *Rock Products*, Sept. 1940.

For these reasons, the study of concrete durability, as affected by the fineness of the cement, should include consideration of the size distribution of the cement and the presence of humps in the grading curve. It is suggested that a cement following the logarithmic grading, in combination with a suitable amount of entrained air, will produce the most durable concrete. It will also provide unreacted clinker for continuing hydration and autogenous healing for a long time to come.

By M. SPINDEL*

Next to the right chemical composition and satisfactory burning of cement clinker it is fine grinding in combination with the necessary percentage of gypsum which has the most important beneficial effect on plasticity, soundness, density, strength and, as a rule, also durability of wet and hardened mortar and concrete. Regarding durability the effect of gypsum content and fineness of cement on shrinkage must be considered. The shrinkage should be determined both during setting and hardening.

Although some deficiencies of mortar and concrete are often due to the high shrinkage-strength ratio of too finely ground cements, those who long for the return of coarse-ground portland cements must remember that deterioration of mortar and concrete in those times was usually the result of the unsoundness of coarse-ground cements. Mills then available could not grind hard raw materials and hard clinker sufficiently fine. Some small cement works in Europe which could not afford better equipment had to close and a new period with much improved grinding mills and methods for making high quality portland cements began which culminated in high early strength portland cement and in high early strength concrete.

Cement and concrete of this type were developed, manufactured and tested in 1913-14 by and under the supervision of the writer. The extraordinary high early strengths of this cement after 2 and 7 days of curing were due mainly to the chemical composition and adequate burning of the clinker but the mill then available at the factory did not produce the fineness of cement considered most suitable. The residue by weight on a standard sieve similar to B.S. 170 mesh test sieve, was more than 15 percent compared to the present maximum of 5 percent in the British Standard Specification.

Although the first high early strength portland cements were coarse-ground, the tests and trials for soundness, strengths and shrinkage, both in laboratory and field, were highly satisfactory. Unfortunately, at the beginning some cement manufacturers were, and may somewhere still be, of the opinion that high fineness of cement could hide unsoundness of the clinker due to defective chemical composition or insufficient mixing and burning. Such cements having a residue of down to only 1 percent on the B.S. 170 mesh test sieve are, of course, not suitable.

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More important was the fact that for some structures, such as large dams and concrete pavements, the shrinkage of concrete had to be reduced by all means possible and therefore the fineness of the cement was limited. The special requirement for such portland cement was that the residue on the B.S. 170 mesh test sieve must not be less than 5 percent or sometimes not even less than 8 percent. These limits were, of course, more or less arbitrary but everybody concerned had to agree that, for certain special types of structures at least, the tendency must not be the finer the better but which fineness is most suitable.

The authors have made an important contribution to solving this very complex question. Some of their diagrams show a distinct maximum of cycles of freezing and thawing for the fineness of 1800 and 2100 sq cm per g for cement A and cement B, respectively, while cement C behaved differently both according to Fig. 4 and 5 of the paper. The authors will agree that such results usually depend on the type of tests carried out and the special circumstances of testing, for example whether tests with cements of different fineness are carried out for equal cement weight per cubic yard and for equal slump and workability, or for equal strengths, whether on dried or wet concretes and manifold other variations in choosing the cements and aggregates and in making, curing and testing the specimens.

Convinced that the authors will continue these very complex but highly important investigations the writer would like to remark only how much he appreciated the microscopical investigations shown in Fig. 6 and 8. It would be of interest to know whether the dye used for this purpose also showed how far the calcium oxide was already converted into calcium carbonate and whether some of the fine cracks were due to this.

The writer's view on this matter given in "Special Cements," Second Congress on Large Dams, Washington, D. C., 1936, may be summarized as follows:

> The cement particles have to be of such a fineness that they can get into and fill the voids between the smallest sand particles. This requires that the diameter of the cement particles should not exceed 1/5th to 1/7th of the diameter of the smallest sand particles. For the right texture of concrete the space for small particles should be reserved mainly for the cement as the binding material and the percentage of fine sand particles passing the B.S. 72 or the B.S. 100 mesh test sieve should be reduced as far as possible. This is specially important if the fine sand is of laminated shape, such as mica. It is true also for similar shaped natural or artificial pozzolanic or lightweight materials.

The fineness of the cement must, on the other hand, be limited in such a way that the hydration of the cement particles, which starts on the surface, does not take place too soon throughout the grains since it has been found that a kernel of unhydrated cement, which will hydrate gradually later, is necessary or at least of great advantage in the process of hardening, especially with regard to low shrinkage.

With both coarse- and fine-ground cements it is essential to work with the lowest water-cement ratio possible and suitable to the equipment and methods

of mixing, placing and curing. In the writer's opinion and according to the experience of other cement and concrete experts, the small particles of cement, although finally hydrated by water, at first tend to be hydrophobic and, therefore, agglomerate in water to larger lumps containing cement, water and air pores. As such lumps of agglomerated fine cement particles are harmful, dispersion of the particles of modern fine-ground cements by mechanical means or by suitable dispersing agents or sometimes both is of great advantage. Some well known dispersing agents have been used successfully for a long time in practice. They, therefore, should be included also in research work on shrinkage and durability of coarse- and fine-ground cements.

It would be of further advantage to test cements of various fineness with aggregates of different fineness modulus containing sand with different percentages of fines passing the B.S. 72 mesh test sieve and the B.S. 100 mesh test sieve and also with a few percent of entrained air partly replacing this fine sand. It is to be expected that further research on the lines suggested will show modern fine-ground cements used with suitable dispersing and plasticizing agents to be far superior to the old coarse-ground cements in every way.

By BRYANT MATHER*

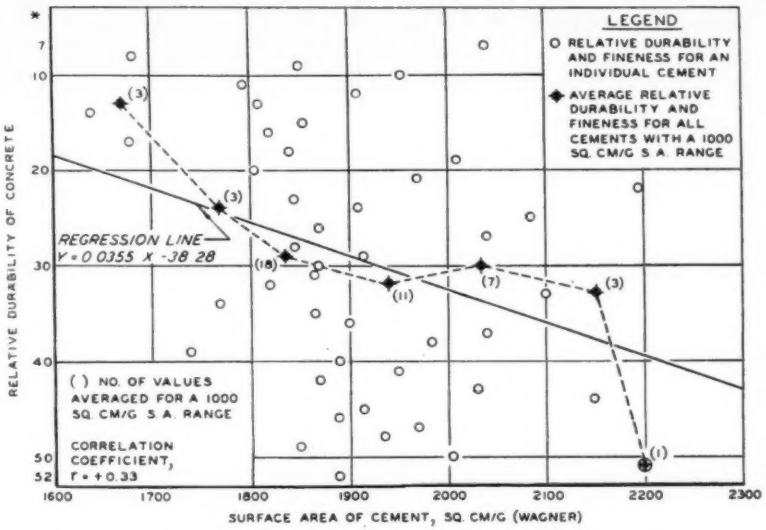
It is very desirable that information derived from carefully controlled laboratory tests be contributed to the consideration of the question of the effect of cement fineness on the properties of concrete. The authors are to be commended for their selection of laboratory testing procedures and for their discussion of the data derived from these tests.

The authors summarize their findings as follows: "The tests described show that concrete made with the coarser cement and subjected to a period of weathering before starting the laboratory freezing-and-thawing cycles is more resistant to cracking and to freezing and thawing than concrete made with fine-ground cement" and "It can be concluded from these test results that concrete made with coarse-ground cement is more resistant to cracking resulting from drying, and that it had a greater resistance to freezing and thawing under normal exposure than concrete made with fine-ground cement." I feel that these two statements present useful, valuable, and well qualified conclusions. I am troubled by the fact that the authors who wrote these carefully framed sentences allowed the paper to be published under the title that was used. I doubt if they believe, as their title suggests that they do, that the durability of all concretes would automatically be improved if the cements used were of lower specific surface.

In 1942 the Central Concrete Laboratory reported† a study of the durability of concrete made with 52 different commercial portland cements. The discussion of the results of this study includes the statement: "The data

*Civil Engineer (Concrete Research), Concrete Research Division, Waterways Experiment Station, Corps of Engineers, U. S. Army, Jackson, Miss.

†"Cement Durability Program—First Interim Report," Central Concrete Laboratory, Corps of Engineers, Mount Vernon, N. Y., June 1942.


LEGEND
O RELATIVE DURABILITY AND FINENESS FOR AN INDIVIDUAL CEMENT
◆ AVERAGE RELATIVE DURABILITY AND FINENESS FOR ALL CEMENTS WITH A 1000 SQ. CM/G S. A. RANGE
REGRESSION LINE
Y = 0 0355 X -38 28
( ) NO. OF VALUES AVERAGED FOR A 1000 SQ. CM/G S. A. RANGE
CORRELATION COEFFICIENT, r = +0.33
RELATIVE DURABILITY OF CONCRETE
SURFACE AREA OF CEMENT, SQ. CM/G (WAGNER)
1600
1700
1800
1900
2000
2100
2200
2300

*THE SIX MOST DURABLE CONCRETES WERE MADE WITH AIR-ENTRAINING CEMENTS, DATA ON THESE ARE OMITTED

Fig. A—Relation of fineness of cement to relative durability of concrete

would indicate that the durability of the concrete made with the cements in this program decreases with an increase in surface area . . . "*. It thus appears that this study of 52 commercial cements used in concrete subjected to very severe natural weathering at Treat Island, Maine, indicated the same relationship between cement fineness and concrete durability that the authors found in their study of three cement clinkers each ground in the laboratory to six different finenesses and tested in concrete in the laboratory.

A more detailed examination of the 1942 data reveals the need for qualifying this indicated relationship. The 52 cements studied included 6 that contained air-entraining additions and since the concrete made with these cements exhibited a degree of durability greatly different from that made with all the other cements, they have been omitted from the following discussion.

Fig. A has been constructed using the data† on relative durability and fineness for the 46 non-air-entraining cements. The trend shown both by the regression line and by the line connecting average points for surface-area class intervals indicates a relationship of decreasing durability with increasing fineness. It is believed however most significant that the coefficient of linear correlation for these data, r, equals + 0.33. This value indicates that a "significant" correlation exists between surface area and relative durability but it also indicates that only about 11 percent of the variation in the relative

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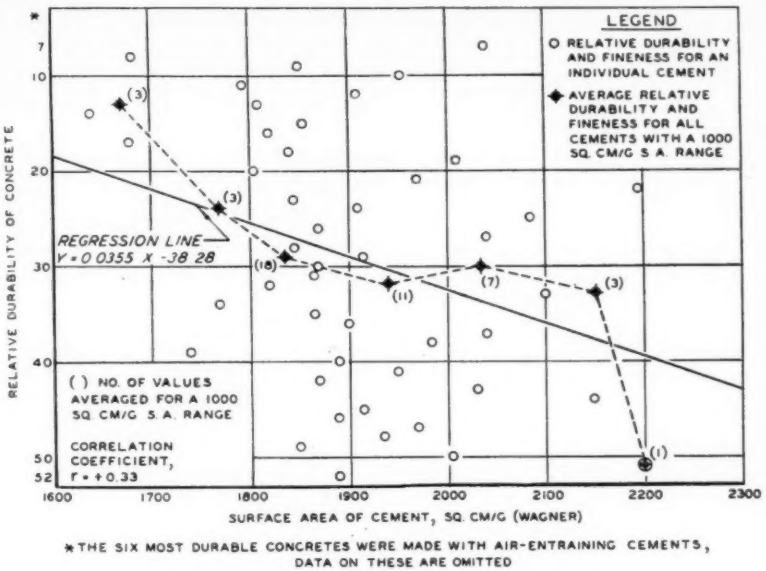

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durability of these concretes can be explained by variation in fineness of the cements used. It also indicates that, for these data, an attempt to predict the relative durability of the concrete using information on the fineness of the cement would be subject to 95 percent of the error that would accompany a purely random prediction.

It is therefore suggested that caution must be exercised in discussing the effects of cement fineness on concrete durability. If all other factors affecting concrete durability could be eliminated or held constant a simple inverse relation might very well be found to exist between concrete durability and cement fineness. Unfortunately however, all other factors cannot be eliminated either in field or in laboratory concrete.

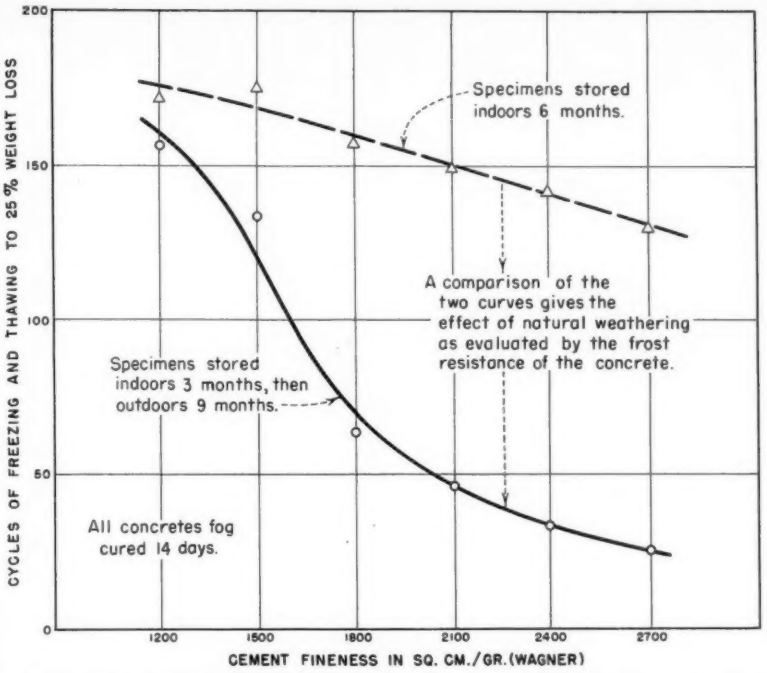
AUTHORS' CLOSURE

Messrs. Woods and Mather have justifiably criticized the wording of the title for its implication that durable concrete can be assured by the use of coarse-ground cement. The authors regret that they did not retain their original title, "The Effect of Cement Fineness on the Durability of Concrete."

Mr. Woods' discussion may itself convey to some readers an implication that the factor of cement fineness is unimportant, since air-entrainment is of greater benefit to the frost resistance of concrete. The terms, "frost resistance" and "durability" are not synonomous, and something more than air bubbles is required to produce a durable, noncracking, concrete. Entrained air is of little benefit to shrinkage, autogenous healing or cement-aggregate reaction. The weathering of concrete, in addition to freezing and thawing, consists of heating and cooling, wetting and drying, and the simultaneous response of the concrete to these factors in terms of shrinkage, plastic flow, cracking resistance, autogenous healing, carbonation, cement-aggregate reaction, etc. These "tearing-down" and "building-up" processes are largely independent of entrained air and the balance between them greatly influences the quality of the matured concrete.

A durability test which attempts to evaluate the net effect of these weathering processes was shown as a supplemental slide at the 1951 San Francisco convention and is reproduced here as Figure B. Both groups of specimens received the same initial curing treatment and since one group was subjected to weathering while the other was stored indoors, their comparison gives the effect of the weathering period on the vulnerability of the concrete to frost action. Both groups of specimens showed declining durability with increasing fineness, but the rate of decline was very pronounced with the weathered concretes. The conclusion from these data is that the concrete containing the coarse-ground cements suffered less damage from actual weathering than the concretes with the finer ground cements. This effect may be due to the greater amount of unhealed internal cracking in the concretes containing the fine-ground cements.

In another cement fineness investigation involving air-entraining concretes, we found that the coarse-ground cements still produced the most



g. B—Natural weathering decreased frost resistance of concretes containing fine-ground cements. Concretes contained Republican River aggregate and Cement *B*

durable concretes. Since the beneficial mechanisms of coarse-ground cement and entrained-air are dissimilar, they should be additive.

Mr. Woods comments that the criterion of 25 percent weight loss is inadequate if not supplemented by expansion measurements. Expansion measurements were obtained in our tests but were omitted from the paper for brevity, since they were consistent with the conclusions drawn from the weight-loss data. These data are shown, in part, in Fig. C. In plain concrete, the 25 percent weight-loss failure usually occurs only after the concrete has manifested considerable expansion. The amount of expansion which has occurred when failure takes place is somewhat influenced by factors such as the age of the concrete, the manner of testing, etc. For example, we found that when we froze and thawed concretes which had first been exposed to weathering for 1 and 2 years, the older concretes crumbled at a lower value of expansion denoting a loss of cohesiveness. In most cases, however, the 25 percent weight-loss data and the expansion data engender the same conclusions.

In our paper, within each series of finenesses of the same cement composition, the SO_3 content was the same. Mr. Woods has pointed out that

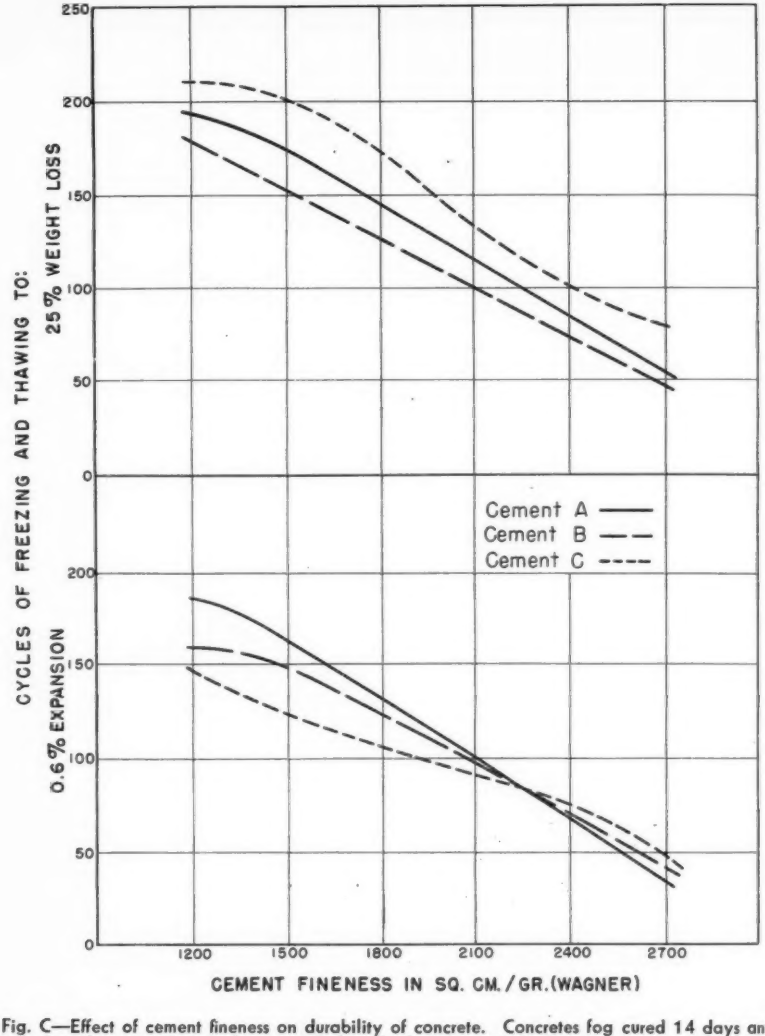


Fig. C—Effect of cement fineness on durability of concrete. Concretes fog cured 14 days and stored outdoors approximately 1 year before the freezing and thawing test

the works of Lerch[1] and Pickett[2] indicate that the optimum SO_3 content with respect to shrinkage depends upon cement fineness; and therefore, the larger shrinkage and cracking noted in our tests for the finer cements may indicate that these cements did not contain enough gypsum. This possibility requires investigation. Another possible effect of SO_3 is that since SO_3 may decrease

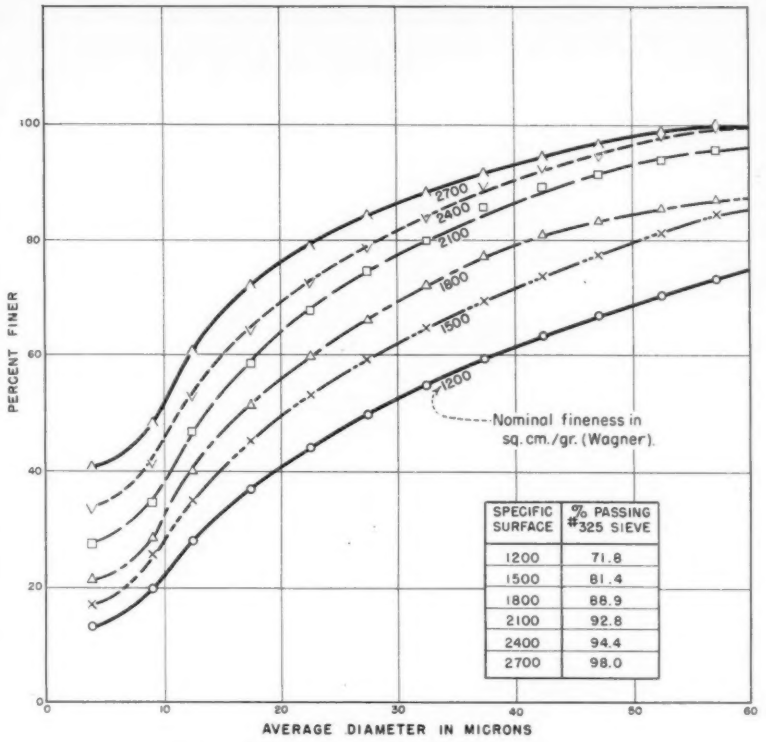


Fig. D—Particle size distribution curves—Cement A

plastic flow,[2] it may influence the relationship between the amount of over-all shrinkage (as determined on a bar) and the amount of internal cracking (observed with a microscope). There are indications that internal cracking may tend to reduce the amount of over-all shrinkage. Tests reported by Meissner[3] show that the quantity of SO_3 required for minimum shrinkage is more than desirable to produce maximum frost resistance in the concrete. These relationships suggest the possibility that the SO_3 content, in certain ranges, may reduce the over-all shrinkage by increasing the amount of internal cracking. This possibility, remote as it may seem, should not be ignored.

Mr. Anderegg suggests that a study of concrete durability as affected by the fineness of the cement should include consideration of the size distribution of the cement and the presence of "humps" in the grading curve. The grading curves of the cements used in our tests indicated a hump size at about 5 microns (Fig. D). Although the gradation is important, it is doubtful that the interference with the packing caused by this hump size could have caused

all the results described in our paper. The Wagner fineness is inadequate when used to compare cements ground in different mills, but since all our cements were ground in the same mill, their gradations were similar and the effect of gradation would be largely cancelled.

The remarks of Mr. Spindel, concerning the influence of the test procedure upon the test results, illustrate the need for more reliable test methods. This fact is stressed in the paper which showed that the advantage of coarse-ground cement became more apparent when the conditions of the test methods approached the conditions of natural weathering. Misleading test results are inevitable when the basic factors of weathering are ignored in favor of a highly accelerated pseudo-weathering test.

In answer to Mr. Spindel's inquiry as to whether the dye used indicated the depth of carbonation or any cracking due to carbonation, it can be stated that the Patent Blue dye which was used did not clearly indicate the depth of carbonation. A dilute solution of phenolphthalein in water and alcohol may be used for this purpose. In regard to cracking, we have little evidence that carbonation produces cracking. Instead we have found that microscopic shrinkage cracks will frequently fill with calcium hydroxide which is later altered to calcium carbonate in the vicinity of the exposed surfaces of the concrete. Our tests (unpublished) indicate that concrete is more durable when carbonated and that concrete containing coarse-ground cement carbonates more readily.

M. Spindel predicts that if fine-ground cements are used with suitable dispersing agents and with optimum amounts of fine sand and entrained-air, they will be superior to coarse-ground cements *in every way*. If test data do exist in support of this assertion, their references should have been listed.

Mr. Mather has cautioned against predicting the durability of a concrete from the fineness of the cement, due to the influence of the many other factors affecting durability. The influence of these factors require further definition, particularly in regard to their effect on cracking. The single factor of cement fineness remains of major importance to the *cracking* of concrete. It is hoped that the additional knowledge acquired from future work will eventually yield a better perspective of the *relative* importance of the factors affecting the quality of concrete.

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Title No. 47-26

Any laboratory can study prestressed concrete with the simple equipment developed by the authors.

Simple Equipment Economically Explores Prestressing*

By MARVIN L. MASS† and JACK R. JANNEY‡

SYNOPSIS

A tubular grip utilizing type metal to hold the prestressing wires is described. Other equipment includes a framework against which the prestressing force is applied and adjustable formwork for experimental beams. The equipment is so simple that it can be easily and inexpensively built to permit study of prestressed reinforced concrete in the small laboratory.

INTRODUCTION

During the past few years tremendous interest has been aroused in prestressed reinforced concrete. Many a university and testing laboratory has shunned the field because of the supposed cost of the equipment, *i.e.*, abutments, hydraulic jacks, etc. It has been found that most of the experiments can be performed with simple equipment. A new type of grip brings prestressed reinforced concrete experimenting within the range of any small laboratory. The purpose of this article is to show what can be done with a minimum of financial backing, and to encourage other institutions to tackle some of the problems that need investigating.

EQUIPMENT

The basic equipment needed is as follows: a framework to anchor the wires to, grips to hold the wires, a mechanical or hydraulic jack (not necessary if tubular grips are used) and lumber for formwork.

Framework

The pre-tensioning method requires heavy abutments while the post-tensioning method may or may not require abutments because in the pre-tensioning method the supports must be capable of taking the large forces

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†Member American Concrete Institute, Assistant Professor of Civil Engineering, University of Colorado, Boulder, Colo.

‡Member American Concrete Institute, Assistant Development Engineer, Portland Cement Assn., Skokie, Ill.

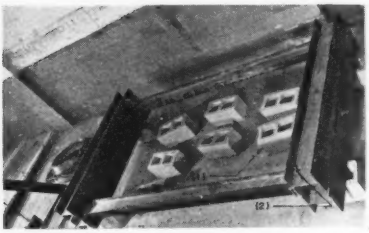

Fig. 1—Concrete blocks were used to support the bottom of the forms (1). Wide flange beams (2) were slotted to provide for any future change in size of wire or grips.

created by the many prestress wires. A framework of steel I-beams and wide flange sections can be used as a buttress for the prestressed wires (Fig. 1-3).

Wire grips

Wedge type—A number of wire grips were devised using the Magnel and Freyssinet principles, but they were too clumsy to handle. Prestressing the wire by jacks was also clumsy, and the tool steel and precision machine work to make them proved expensive.

Tubular type—The wire grip used was a hollow tube threaded and plugged at one end. One end of a wire section was placed in the tube which was then filled with molten type metal (85 percent lead and 15 percent antimony). When the type metal had cooled and hardened the wire was held firmly in the tube (Fig. 4).

It was found most advantageous to deform with a chisel the section of the high tensile wire that was placed in the tube. It was also found necessary to keep the tubular grip hot with a blow torch while pouring the type metal to make sure the tube was completely filled. By letting the type metal cool slowly, instead of rapidly cooling it in water, the gripping action was increased at least three-fold.

A 6-in. type-metal tubular grip was tested with a 0.1875-in. diameter wire and failure (slippage) took place at 3300 lb. When a similar grip was loaded over a period of 48 hours with a 3000 lb load it lost about 25 percent in the first 24-hour period. This loss may have been caused by flow of the type metal, creep of the wire, or creep of the testing machine. When the gripping length was increased to 8 in. and an assembly placed in the framework with a 3300 lb load applied and left for a week there was no appreciable loss in stress.

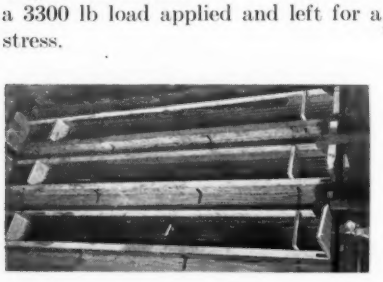

Fig. 2—Wires have been stressed and forms are in place ready to receive concrete

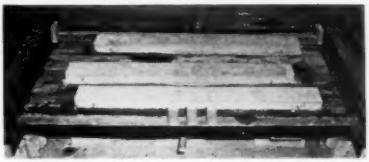

Fig. 3—Forms have been stripped from cast beams. Wires are ready to be released from the grips which anchor them to the framework

The complete wire assembly consisted of wire, two tubular grips, two washers and two nuts. The tubular grips were ¾ in. outside diameter, 12 in. long and had 4 in. of thread.

APPLICATION

Placing wires

The wire assemblies were spaced 1 in. apart in the framework by heavy steel washers. Each wire was drawn up as tight as possible by hand and then a strain gage was placed on the wire and tension was applied by drawing up on the nuts. Approximately eleven turns were required on each nut to obtain the desired tension of 120,000 psi. The strain gage was left on overnight to check for slippage. The strain gage showed little loss of strain in the wire. To make sure all the wires were stressed approximately the same amount they were tuned. Adjustments were made by turning the nuts slightly.

Forms

The formwork was made so the bottom could be raised or lowered to obtain the eccentricity derived from the theoretical beam design (Fig. 3).

Release of wires from abutments

When the concrete beams were cured and dried a sufficient length of time, the prestress wires were released from the abutments by removing the nuts from the threaded tubular grips. The nuts were loosened slowly to equalize the force as it was transmitted into the beams. About three complete turns of the nut were required to entirely release the wire.

While the tension in the wires was being transferred to the concrete, some bond was lost in the end of the beam. To determine the bond slippage near the end of the beam the wires were painted with finger nail polish at the point where they emerged from the beam and a small scratch was placed in the polish. The scratches were observed while the wires were being released from the abutments. The apparent slippage of wire into the concrete varied slightly with each wire, but the maximum amount of slippage in any

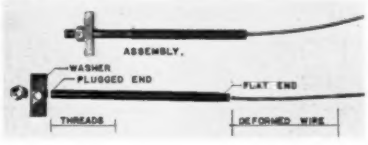

Fig. 4—Tubular grip

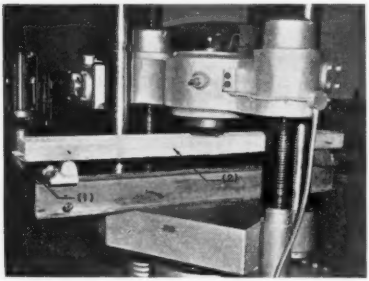

Fig. 5—Testing beam. (1) Cut wire ends. (2) Thread for measuring deflection

wire was very small. This was not considered serious. When all the tension from the wires was taken by the concrete, the grips were cut with a hack saw.

Beam tests

The beams were loaded with a concentrated load at the center, and observations were made to attempt to verify the theoretical design procedure for thin prestressed reinforced concrete beams (Fig. 5).

CONCLUSIONS

With the apparatus described almost any school can instruct students in prestressed reinforced concrete, and can perhaps conduct some of the research that is so urgently needed. There are many projects presented in each paper or book written on the subject. The major conclusion is that there is a lot of work to be done in the field of prestressed reinforced concrete.

As a guide, the following projects are thought important by a number of interested groups.

1. Investigation of fatigue strength.
2. Development of better anchoring devices.
3. Strain gage measurement of small diameter wire.
4. Check losses in prestress due to shrinkage and creep.
5. Bond stresses in the pre-tensioning method.
6. Bond strength in relation to the size of aggregate.
7. Investigation of static and impact strength of prestressed members.
8. Combination of concrete strengths and tensions in the wire.
9. Shearing strength.
10. Combined bending and compression.

Tied splices of deformed bars in which lugs are interlocked indicate efficiency equal to spaced splices.

Laboratory Tests of Spaced and Tied Reinforcing Bars*

By WILLIAM T. WALKER†

SYNOPSIS

Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little or no advantage would be obtained by spacing deformed reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which interlocking of lugs could take place there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Twelve beams containing three different types of deformed bars either spaced or tied and extending the entire length of the specimens were tested. The sixty vertical pull-outs tested also contained three different types of deformed bars. Half of these were made with the bars spaced and half with the bars tied together. The reinforcing arrangement brought only one bar out of the specimens for pulling; the other was allowed to bear on the base plate.

Spacing of the bars in both beams and pull-outs conformed to the Joint Code minimum spacing requirement of 2½ diameters center to center with a minimum clear distance of 1½ times the maximum size of coarse aggregate.

INTRODUCTION

In structures, reinforcing bar splices may be either "spaced" or "tied." Past practice has usually required that an allowance be made for a reduction in bonded area for tied splices. This was taken care of by lengthening the splice. The present tests were intended to provide data on the reduction in effective bond area resulting from wiring bars together.

The large number of specimens tested resulted in average data of sufficient reliability to permit the conclusion that tied splices of highly deformed bars are at least equal in efficiency to spaced splices when lap lengths are equal and when load and specimen conditions approximate those of the tests.

The data from these tests alone are not sufficient for positive assertion that tied splices are preferable under all field conditions. They do, however,

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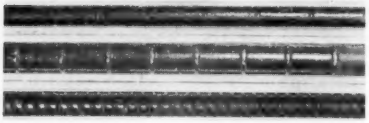

Fig. 1—Types of round bars used as reinforcement for beams. Top—Type 1, ¾-in. diameter. Middle—Type 2, 1-in. diameter. Bottom—Type 3, ¾-in. diameter.

check the results of tests by others[1, 2] on beams and columns which show that the difference in splice strength would normally not be large.

SPECIMENS

The beam specimens were divided into two strength groups, I and II, of six beams each to permit comparison of variations due to differences in concrete strengths. Group I beams were provided with blockouts, exposing the steel for strain measurements, while Group II had no blockouts.

The beams of Group I were made of high strength concrete, using a 1:3.82 mix with a water-cement ratio of 0.40 and a cement content of 8.25 sacks per cu yd. Group II beams were made of lower strength concrete, having a mix of 1:5.0, water-cement ratio of 0.55, and a cement content of 6.5 sacks per cu yd. Both groups were made from Platte River aggregate (maximum size three-quarter in.) with normal cement (Type I) plus 3 percent calcium chloride. They were fog cured and tested at 28 days.

Three types of reinforcing bars were used for the beams, as shown in Fig. 1. The bar designated as Type I was a wartime product so slightly deformed that it was probably not much superior to a plain bar. The spaced and tied bar assemblies are shown in Fig. 2. One spaced and one tied beam were made for each type of deformed bar in each of the two groups. The spaced bars were a clear distance of 1½ diameters apart. Size of the reinforcing steel and other pertinent data are given in Table 1.

It will be noted that these beams do not contain splices. The intent was to investigate, through comparison of stress, deflection, end slip, and ultimate load, possible loss of bond resulting from tying the bars together.

The beams were all 8 x 8 x 48 in. The blockouts in Group I were 2 x 2¼ x 8 in. and were 13 in. from each end of the beam. Strains in the exposed steel were measured by SR-4 gages cemented to the sides of the reinforcing bars. The supports were 2 in. from each end giving a span of 44 in. The loads were applied at the quarter points 11 in. from the supports.

The concrete mix for the pull-outs was 1:1.93:2.42 by weight with a water-cement ratio, by weight, of 0.50. The aggregate was Platte River gravel and Bertrand sand, with the gravel ranging from No. 4 to ¾ in. A Bureau laboratory-blended modified cement (Type II) was used.

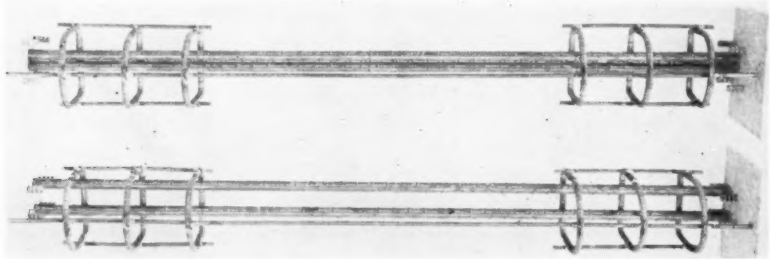

Fig. 2—Spaced and tied reinforcing bar assemblies (Type 2 bar). Stirrups were not used in Group I beams.

The reinforcing bars used in the pull-outs are shown in Fig. 3. The round standard deformed bar had a yield point of 46,800 psi, the Bethlehem bar 42,400 psi, and the square bar 46,500 psi. The Bethlehem bar deformations conformed to ASTM Designation A 305-49. The standard deformed bars did not conform to this designation.

All pull-out specimens were 8 in. in diameter with a steel embedment of 8 in., being cast in the direction of pull. The bars were arranged as shown in Fig. 4.

TESTING

All the tests were performed in a 300,000-lb universal hydraulic testing machine. For beams, end slip was measured to 0.0001 in. at both ends of each bar, and readings were taken at each 2000-lb load increment. Steel stresses were measured at each load increment for Group I beams and center deflection at each increment for Group II. The progression of cracking was observed and marked on all beams (Fig. 5 and 6).

The dial gage arrangement for measuring steel slip in the pull-out specimens was that commonly used; *i.e.*, one dial reading slip at the free end and two dials recording slip at the loaded end. For the spaced-bar specimens, the continuous or pulled bar was considerably off center, and a special bearing plate was used to give equal bearing areas on each side of the pulled bar. This bearing plate allowed some concrete on the long off-center side to be ineffective in bearing, but not in such a way as to appreciably affect the value of spaced versus tied bar specimen results for comparative purposes.

Pull-outs were loaded continuously with readings taken at 1000-lb increments. Loads at first slip and also at the yield point of the steel were noted.

TABLE 1—BEAM SPECIMEN DATA

Beam No.	Ultimate strength, 6 × 12-in. cylinders, psi	Modulus of elasticity, concrete, $E_c \times 10^6$	n
Group I			
1-AT	7950	4.05	7.40
1-AS	7710	3.90	7.69
2-AT	8020	3.90	7.69
2-AS	8280	3.85	7.79
3-AT	7385	3.80	7.88
3-AS	6990	3.80	7.88
Group II			
1-BT	5485	3.95	7.60
1-BS	5280	3.62	8.30
2-BT	4545	3.67	8.20
2-BS	5055	3.49	8.60
3-BT	4990	3.33	9.00
3-BS	5085	3.67	8.20

Key: 1—Transverse lug type reinforcement bar, ¾-in. ϕ (see Fig. 1)
2—Standard deformed type bar, 1-in. ϕ (see Fig. 1)
3—Helical rib type bar, ¾-in. ϕ (see Fig. 1)
A—Group I—Blockouts, no stirrups. Age—28 days
B—Group II—Stirrups, no blockouts. Age—29 days
T—Tied bars
S—Spaced bars

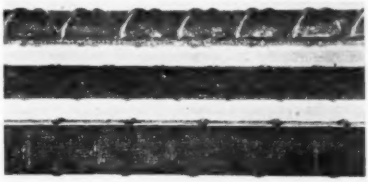

Fig. 3—Reinforcing bars for pull-outs. Top to bottom: ¾-in. round Bethlehem bar, ¾-in. round standard deformed bar, and 1-in. sq standard deformed bar

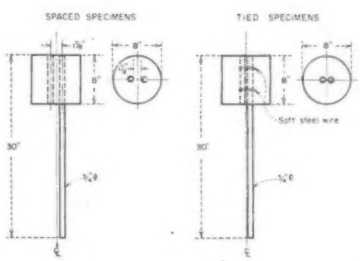

Fig. 4—Position of reinforcing in spaced and tied specimens

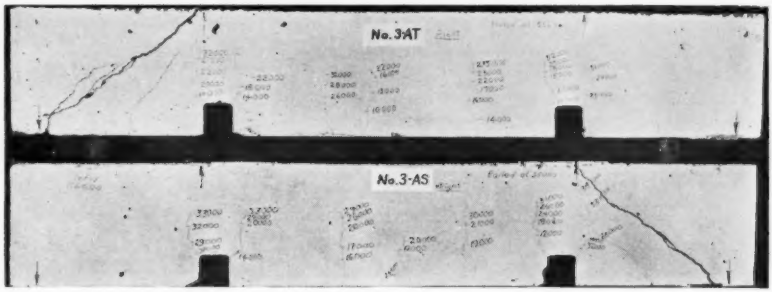

Fig. 5—Group I beams after testing to failure with quarterpoint loads. Note typical diagonal tension failures

Fig. 6—Group II beams after testing to failure with quarterpoint loads. Typical bond failures.

The steel arms which formed the bearing points for the gages at the loaded ends of the specimens were attached to the steel about 1 in. below the concrete. Elongation of the steel in this length was ignored.

RESULTS

The beam specimens with tied bars seemed to have a tendency at high loadings to show more end-slip than those with spaced bars. This is particularly evident from the average end-slip curves of Fig. 7.

In general, beams reinforced with tied bars failed at slightly lower loads than those with spaced bars (see Table 2), although the differences were scarcely significant except in beams 1–AT and 1–BT reinforced with slightly deformed bars. The greatest amount of slip also occurred in these beams, and they were the only ones in which ultimate failure was due to loss of bond.

There was good agreement between the curves of measured stresses in spaced and tied bars for all the beams in Group I (Fig. 8). These stresses, however, were considerably lower than the computed stresses. The load-stress curves indicate that the average steel stress was slightly higher in beams reinforced with spaced bars than in beams reinforced with tied bars. This checks with the general tendency of the spaced bars to show less center deflection (Fig. 9) and less end slip (Fig. 3) indicating that spacing was slightly more efficient than tying.

TABLE 2—AVERAGE SLIP AND DEFLECTION MEASUREMENTS

TABLE 2—AVERAGE SLIP AND DEFLECTION MEASUREMENTS

Beam No.	Maximum load, lb	First crack		Slip at 10,000 lb		Slip at 20,000 lb		Slip at 25,000 lb		Slip at 30,000 lb		Deflection at 25,000 lb		Number of cracks	Type of failure
		at blockout	outside of blockout	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.	Max.	Ave.		
1-AT	27,000	14,000	15,000	0.00064	0.00034	0.00100	0.00072	0.00256	0.00167	———	———	———	———	4	Bond
1-AS	34,000	15,000	12,000	.00018	.00013	.00049	.00031	.00059	.00040	0.00087	0.00056	———	———	5	Bond
2-AT	37,000	36,000	26,000	.00025	.00019	.00039	.00039	.00059	.00050	.00080	.00066	———	———	4	Diagonal shear
2-AS	39,000	37,000	24,000	.00048	.00035	.00088	.00060	.00108	.00076	.00126	.00092	———	———	3	Diagonal shear
3-AT	33,000	10,000	12,000	.00038	.00029	.00105	.00067	.00129	.00075	.00170	.00112	———	———	7	Diagonal shear
3-AS	34,000	10,000	12,000	.00039	.00032	.00073	.00038	.00093	.00075	.00110	.00091	———	———	7	Diagonal shear
1-BT	28,900	12,000		.00011	.00008	.00045	.00027	.00166	.00095	———	.00088	0.0507	0.0492	4	Bond
1-BS	34,800	14,000		.00015	.00008	.00029	.00020	.00045	.00031	.00120	.00088	.0480	.0447	5	Bond
2-BT	32,000	14,000		.00011	.00005	.00024	.00019	.00058	.00039	.00198	.00141	.0468	.0450	6	Diagonal shear
2-BS	31,000	14,000		.00009	.00005	.00024	.00016	.00033	.00024	.00079	.00063	.0482	.0482	6	Diagonal shear
3-BT	35,000	14,000		.00013	.00008	.00028	.00014	.00043	.00025	.00098	.00076	.0508	.0504	8	Diagonal shear
3-BS	34,000	12,000		.0009	.00007	.00024	.00018	.00036	.00027	.00060	.00042	.0482	.0480	7	Diagonal shear

Fig. 7—Average of end slips for beams with tied and spaced reinforcing. Three types of bars used for each condition: transverse lug, standard deformed and helical rib

In general, agreement between the center deflection curves of the beams in Group II as regards the spaced and tied arrangements, as shown in Fig. 9, was close. Each curve changed in slope at the point in loading where cracking began; deflection increased more rapidly for succeeding load increments.

For the pull-outs, average results for spaced and tied standard deformed bars are shown in Fig. 10. Looking at the free end slip values, it will be noted that slipping started near the same load for each type splice, but advanced much more rapidly for the spaced bar specimens as the load increased. Bottom slip values were approximately the same for both type splices until a bond of 300 psi was reached, after which the tied bar splices showed less slip.

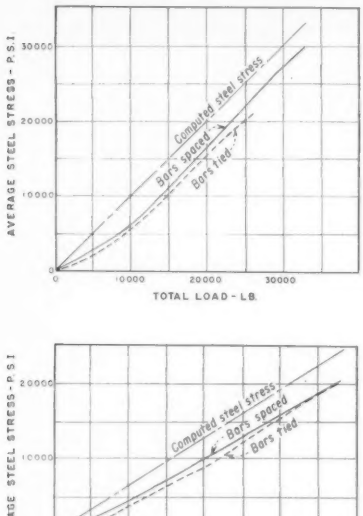

Fig. 8—Load-stress diagrams. See Table 1 for description of bars

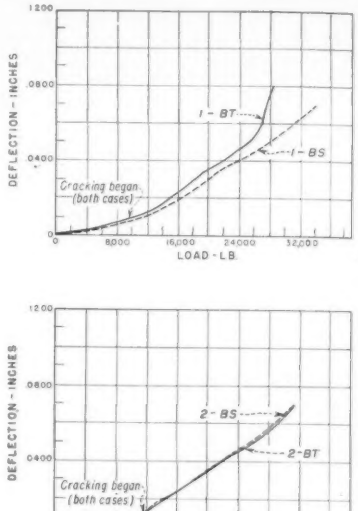

Fig. 9—Load-deflection curves. See Table 1 for description of bars

Fig. 11 shows average results for Bethlehem bar splices. Here again, free end slip values at low loads are about the same for both tied and spaced splices. Then for bond values about 500 psi, the spaced specimens begin to show definitely greater free end slips which continue to be greater until failure.

Fig. 12 shows similar comparative data for the square bars. Differences in bond values between the two types of splices vary in much the same manner as for round standard deformed bars.

In the present study the tied specimens were of primary interest. Several were broken apart to determine the extent of grout penetration between the bars and the degree of mechanical interlocking. Fig. 13 and 14 show good penetration of grout between both round and square bars. There was no evidence that the grout between the bars was weaker than the concrete around the nonadjacent surfaces nor that the grout possessed poorer bonding properties.

The splice lap used in these pull-out specimens was shorter than would be called for in ordinary design practice. Tests by Abrams, Talbot, and others show that unit bond strength decreases with increased length of embedment, but only by a slight amount. Data from several beam and pull-out tests show that bond values on the average would be lowered approximately 30 psi for every 10 in. of increased embedment. An embedment of 8 in. probably gives a slightly higher than average bond value.

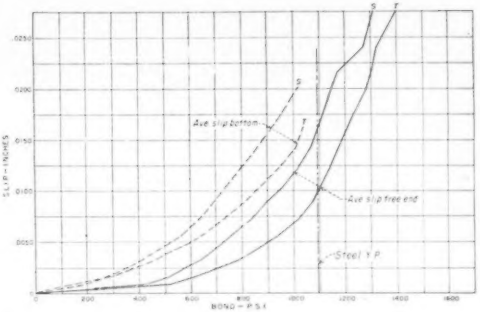

Fig. 10—Average bond-slip curves for standard deformed bar. Ten specimens each

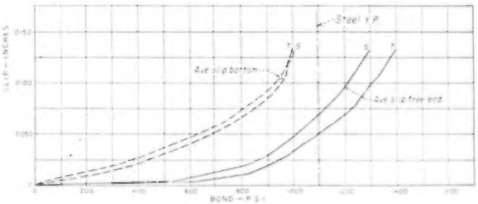

Fig. 11—Average bond-slip curves for Bethlehem bar. Ten specimens each

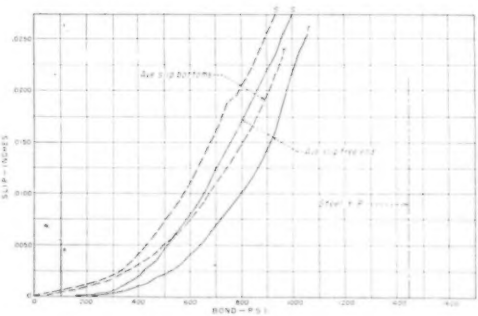

Fig. 12—Average bond-slip curves for sq deformed bar. Ten specimens each

Table 3 compares test splice bond values of the three types of commercial deformed bars for certain specific slips.

CONCLUSIONS

1. Beam tests showed no significant difference between the two splicing methods, though at higher loads there was some indication that spaced splices might be slightly preferable.

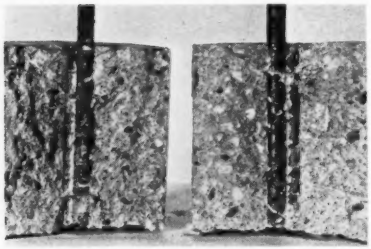

Fig. 13—Round bar specimens split apart to show penetration of grout between bars

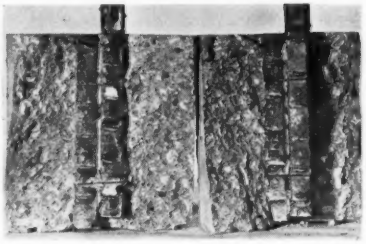

Fig. 14—Square bar specimens split apart to show penetration of grout between bars. Some grout inadvertently broken out in handling

TABLE 3—COMPARISON OF PULL-OUT SPLICE TEST RESULTS*

Slip values, in.†	Bond values, psi		Steel stress, psi	
	Spaced splices	Tied splices	Spaced splices	Tied splices
Standard deformed bars—¾-in. ϕ				
0.001	292	318	12,500	13,600
0.01	822	981	35,100	42,000
Bethlehem bars—¾-in. ϕ				
0.001	318	371	13,600	15,850
0.01	1000+	1000+	43,000+	43,000+
Square deformed bars—1-in. □				
0.001	266	292	8,500	9,350
0.01	617	736	19,700	23,600

*Embedments for all pull-outs—8-in.
†All bar slips are averages of top and bottom values.

2. In beams the semideformed bars indicated a more pronounced difference in favor of spaced splicing than the two more highly deformed bars.

3. The pull-out tests indicated that there was no weakening of bond at the tied splices due to the proximity of the deformed bars when the splices were vertical and the concrete was well vibrated.

4. Evaluation of end slip data from the 8-in. pull-outs showed that on the average the performance of the tied bar arrangement was even better than that of the spaced bar arrangement.

5. Considering the entire data, showing in some tests a slight superiority in the performance of spaced bars and in others a slight superiority in performance of tied bars, it is concluded that there is no important loss of bond when deformed bars are tied together at splices.

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Proper construction machinery adjustment and timing of finishing operations are important factors in finishing air-entraining concrete pavements.

Finishing Air-Entraining Concrete Pavements*

By CHARLES W. ALLEN†

SYNOPSIS

Based on a survey of state highway department practices, common difficulties in finishing air-entraining concrete pavements are enumerated. Practices in adjusting transverse finishing machines are given and revised finishing practices are discussed. Delayed finishing, as specified in several states, is considered in connection with the riding qualities of the finished surfaces. On the basis of experience in Ohio it is concluded that smooth pavements can be built of air-entraining concrete without delayed finishing.

To present as complete a picture as possible of the precautions used in finishing pavements made of air-entraining concrete, each state highway department was contacted and thirty-seven replied. It was the general opinion that placing and finishing air-entraining concrete pavements requires no more attention than pavements built with normal portland cement.

To produce uniform results with normal or air-entraining concrete all the ingredients in the mix must be proportioned with accurate batching equipment. Where air-entraining admixtures are used, adequate and satisfactory batching equipment is desirable for introducing the admixture. The Colorado Highway Department reports irregular results when the air-entraining agent was added into the skip or drum from a cup.

This department also reports that air-entraining admixtures mixed on the job are sometimes ineffective, especially in cold weather. Admixtures stored over night in cold weather did not give satisfactory results. A new batch mixed with warm water gave the desired results.

Assuming that the mix has been properly designed and mixed, air-entraining concrete will have a number of characteristics slightly different from those of normal concrete. Air-entraining concrete is more plastic, cohesive and considerably more sticky than normal concrete, especially in the mixes usually used for pavements. This sometimes results in a tendency for the concrete to tear under the screeds of the finishing machine. At other times the screeds leave a pock-marked surface. These conditions are more pre-

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valent on the drier mixes used for vibratory finishing than on the wetter mixes used for ordinary finishing, the number and size of the pock marks depending on the slump of the mix and the atmospheric conditions. As a general rule, tearing and pock marking of the surface can be eliminated by increasing the rate of oscillation of the screeds without increasing the forward speed of the finishing machine. In some instances, with higher sand and water content, pock marks cannot be so removed but can be removed with a heavy straightedge or hand float.

SETTING SCREEDS OF TRANSVERSE FINISHING MACHINE

A rebound or rubbery condition of air-entraining concrete has sometimes occurred behind the screeds of the finishing machine. To overcome this condition, the transverse screeds are adjusted to a lower tilt than is generally used on normal concrete. Good results have been obtained by giving the front edge of the first screed a lift of approximately $\frac{1}{8}$ in. The rear screed is kept in a level position. The Kansas State Highway Commission reports that on normal concrete the front screed is adjusted $\frac{1}{8}$ in. high and the rear screed $\frac{1}{16}$ or $\frac{1}{32}$ in. high to leave enough material for the longitudinal float. On air-entraining concrete this department found it expedient to adjust both screeds as close to the actual cut-off grade as possible.

The amount of rebound of air-entraining concrete can in many instances be controlled by varying the head of concrete carried by the screeds. As a general rule good results are obtained when the head in front of the first screed has a uniform depth of not less than 3 in. nor more than 6 in.

TIMING OF FINISHING OPERATIONS

In the opinion of most highway departments the switch to air-entraining concrete has required a change in the timing of finishing operations. With normal concrete the finishers depended on water bleeding out of the mix to nourish the surface and aid in finishing operations. The paving and finishing operations were spaced far apart. This permitted long passes with the finishing machine and kept the hand finishers far behind the paver.

With air-entraining concrete bleeding has, in most instances, been eliminated and the hand finishers must work with the original concrete with little or no additional lubrication due to bleeding. Most state highway departments believe that with air-entraining concrete it is necessary to keep hand finishing as close to the paver as possible. It is necessary to have the mix wet enough to allow finishing operations to proceed without undue difficulties. For many years in Ohio the specified slump range for pavement concrete, finished without vibration, has been 2 to 3 in. With air-entraining concrete slumps are near the upper limit.

The Connecticut Highway Department keeps the longtiudinal finishing machine approximately 100 ft behind the transverse finisher to allow for settlement after the transverse finishing machine passes over the concrete. The relationship between the capacity of the mixing equipment and the

finishing equipment is most important. The controlling factor should always be the finishing.

Kansas reports that, even with all operations kept as close to the paver as possible, tearing frequently occurs on hot windy days during the passage of the longitudinal finisher or bull float. The remedy there is to sprinkle the surface with a light spray of water, just ahead of the float. This state reports that this has been done in several instances with satisfactory results and no after damage (up to two years of observation).

One other state, Minnesota, makes provision for sprinkling the surface before the final finishing operation. The procedure will be discussed later.

The majority of states, including Ohio, using air-entraining concrete have not found it necessary to spray the surface with water before final finishing operation to avoid tearing of the surface.

Iowa reports that its specification for finishing concrete pavements calls for two beltings of the pavement with 8-in. canvas belts. In most instances this belting is a hand labor operation. On some municipal jobs, the resistance offered by the belt on 24-ft slabs was more than two workmen could overcome. To get a surface finish in some cases a 4-in. rubberized belt was used. On other projects the desired finish was obtained by folding back the standard canvas belt about 3 in. so that only about 5 in. was used.

The Colorado Highway Department reports that considerable care must be used in the brooming operation, "as the surface of air-entraining concrete will tear easily." "A heavy coarse broom carelessly handled will cause tearing and roughing of the surface." A soft broom has been used with some degree of success but there is a tendency to leave too much water in it.

RIDING QUALITIES

Motorists notice most the riding qualities of a pavement. A smooth riding pavement, in their opinion, is a job well done. The finishing of air-entraining concrete has required modifications in the finishing technique as noted above and for that reason has rightly or wrongly been blamed for poor riding qualities.

In a Highway Research Board paper, "A Study of Delayed Finishing of Concrete Slabs," J. H. Swanberg reported an investigation to determine whether increasing the elapsed time before the second screeding operation would improve the placement of air-entraining concrete. A major item of concern was the poor riding surfaces on some of the projects.

One of the conclusions drawn was that "The finishing operations can be delayed longer in case of concretes made with blended cement and air-entraining cement than with standard portland cement and the vibrated blended cement concrete". The average slumps for the various types of mixes were: standard—2.22 in., air-entraining—4.26 in., blended—4.58 in. and blended-vibrated—1.52.

Field results in 1947, after delayed finishing was adopted, showed that the projects had better riding surfaces than the 1946 projects.

The Minnesota specifications have been revised to require longitudinal

floating not earlier than one hour after the concrete has been deposited. As an aid to proper operation of the float under adverse weather conditions, it is equipped with a water supply and nozzles capable of producing a fine mist or spray. The pavement surface is sprayed only when drying is unusually rapid, and then only to the extent necessary to provide proper surface moisture conditions for the floating and the succeeding straightedging.

The California Highway Department has for many years used delayed finishing for normal cement concrete pavements. In this state air-entraining concrete has been limited to one project. On this project it was necessary to keep the finishing much closer to the mixer than with the ordinary mix.

Five other states commented on riding characteristics. Two states, Rhode Island and Kansas, reported that the riding characteristics of air-entraining concrete pavements were not considered as good as those of pavements made with normal cement or that it was more difficult to secure good riding surfaces when air-entraining concrete was used. Three states, Idaho, Michigan and Utah, reported that smoothness varies with projects. The consensus was that this was due to workmanship and not to the air-entraining concrete.

Since no mention of riding characteristics was made by the other states, it is assumed that satisfactory results are being obtained.

Based on Ohio's own experience, there is no reason to believe that the surfaces obtained with air-entraining concrete are not as good as those obtained with normal concrete. A few months ago the permissible surface tolerance was changed from ¼ in. to ⅛ in. from a 10-ft straightedge applied parallel to the centerline of the pavement.

For many years it has been Ohio practice to test the smoothness of the surface in one or more lines with a profileometer. Variations in excess of the specified tolerances are removed at the contractor's expense. During 1949 and to date in 1950, 68.5 miles of pavement have been completed and checked. The specified tolerance on 54 miles was ¼ in. in 10 ft. On the remaining 14.5 miles the specified tolerance was ⅛ in. in 10 ft.

On the 54 miles of pavement with a specified tolerance of ¼ in. there were 289 marks in 217.9 line miles or an average of 1.3 marks per line mile. Marks per line mile for the 13 projects ranged from 0.0 to 11.3.

On the four projects built with a specified tolerance of ⅛ in., there were 1905 marks in 60.4 line miles. Eighteen hundred and forty-nine marks were found on two projects built by the same contractor, representing 61 percent of the line mileage. On one project, 8.063 miles long, approximately 1.5 miles were built late in 1949. The pavement was completed during 1950. The majority of the 1450 marks found on the project were in the short section built in 1949 before changes in personnel and equipment.

If these two projects are disregarded and the two projects built to a ⅛-in. tolerance, although the specified tolerance was ¼ in., are included, the number of marks per line mile would be 1.0.

On the basis of these data it can be concluded that very smooth pavements can be built with air-entraining concrete without resorting to delayed finishing.

The failure of the Tacoma Narrows Bridge focused the attention of engineers on the importance of considering the action of wind forces on suspension bridges. Here the author studies similar forces on braced and unbraced arch ribs and shows their effect on moments and shear.

Effects of Lateral Loads on Arches*

By JAMES P. MICHALOS†

SYNOPSIS

The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shear in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced with struts normal to the ribs. Numerical studies are included.

INTRODUCTION

In general, an arch is designed for gravity loads, and the designer may check the structure for the effects of lateral loads. This does not mean that these effects are unimportant. It has been stated‡ that in the 426-ft Ammer Arch Bridge, near Echelsbach, Germany, the wind stresses are about 40 percent of the total stress, including dead load, live load and temperature.

This paper§ deals with the effects of wind on unbraced arch ribs and on arch ribs braced with very flexible or very stiff struts normal to the ribs. Both qualitative and quantitative studies are made of moments and shears. No quantitative studies are made of deflections.

UNBRACED RIBS

First, an influence table and influence lines are presented for parabolic arches subjected to lateral loads. These arches are of constant cross section, but several ratios of rise to span and torsional to bending stiffness are included. Second, a procedure is introduced for drawing curves of moments

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‡Gerhart, Rudolf, "Die Hochbrucke uber die Ammer bei Echelsbach," *Beton und Eisen*, No. 6, 1930.

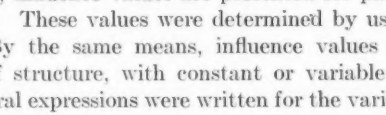
§This work is based on "Analytical Procedures for Structures in Space and a Study of Wind on Arches," by James P. Michalos, a dissertation presented the Graduate School, Northwestern University, Evanston, Ill., in August 1949, in partial fulfillment of the requirement for the degree Doctor of Philosophy.

along the structure. Third, numerical studies are made. These include a consideration of the possible influence of variation in cross section along the structure—haunching—on the distribution of moments.

Influence lines for parabolic arches subjected to lateral loads

In Table 1, influence values are presented for parabolic arches of constant cross section. These values were determined by using the shear and torsion analogy.* By the same means, influence values could be determined for any shape of structure, with constant or variable cross section, lying in a plane. General expressions were written for the various terms of the equations

TABLE 1—INFLUENCE VALUES FOR MOMENTS AND SHEAR AT THE SUPPORTS OF PARABOLIC ARCHES. The arch has a constant cross section and has a concentrated load normal to its plane.

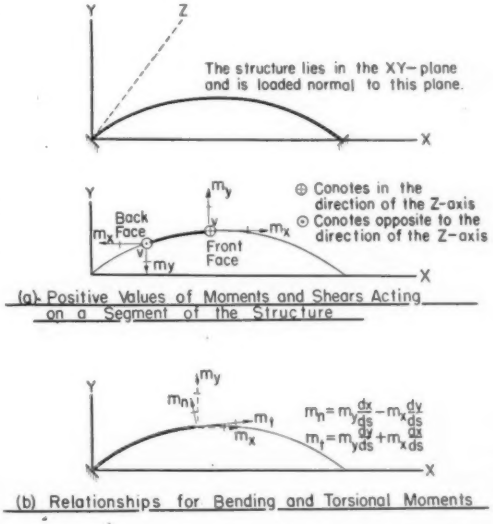


		$\frac{I}{I_1} = 0.5$			$\frac{I}{I_1} = 1$			$\frac{I}{I_1} = 5$			$\frac{I}{I_1} = 10$			$\frac{I}{I_1} = 100$		
$\frac{x}{L}$	Values of a	$-m_{xa}$ in terms of Pb	m_{ya} in terms of PL	v_a	$-m_{xa}$ in terms of Pb	m_{ya} in terms of PL	v_a	$-m_{xa}$ in terms of Pb	m_{ya} in terms of PL	v_a	$-m_{xa}$ in terms of Pb	m_{ya} in terms of PL	v_a	$-m_{xa}$ in terms of Pb	m_{ya} in terms of PL	v_a
0	0.1	0	0.08300	0.97200	0	0.08300	0.97200	0	0.08300	0.97200	0	0.08100	0.97200	0	0.08100	0.97200
	0.2	0	0.12800	0.89600	0	0.12800	0.89600	0	0.12800	0.89600	0	0.12800	0.89600	0	0.12800	0.89600
	0.3	0	0.14700	0.78400	0	0.14700	0.78400	0	0.14700	0.78400	0	0.14700	0.78400	0	0.14700	0.78400
	0.4	0	0.14400	0.64800	0	0.14400	0.64800	0	0.14400	0.64800	0	0.14400	0.64800	0	0.14400	0.64800
	0.5	0	0.12900	0.50000	0	0.12900	0.50000	0	0.12900	0.50000	0	0.12500	0.50000	0	0.12500	0.50000
	0.6	0	0.09600	0.35200	0	0.09600	0.35200	0	0.09600	0.35200	0	0.09600	0.35200	0	0.09600	0.35200
	0.7	0	0.06300	0.21600	0	0.06300	0.21600	0	0.06300	0.21600	0	0.06300	0.21600	0	0.06300	0.21600
	0.8	0	0.03200	0.10400	0	0.03200	0.10400	0	0.03200	0.10400	0	0.03200	0.10400	0	0.03200	0.10400
	0.9	0	0.00900	0.02800	0	0.00900	0.02800	0	0.00900	0.02800	0	0.00900	0.02800	0	0.00900	0.02800
0.33	0.1	0.32149	0.07987	0.68863	0.30990	0.07912	0.69064	0.32267	0.08130	0.69073	0.32431	0.08186	0.69116	0.33005	0.08312	0.69329
	0.2	0.50591	0.12195	0.58812	0.50486	0.12339	0.58902	0.50813	0.12784	0.58974	0.51368	0.13191	0.59062	0.54924	0.15376	0.59390
	0.3	0.58097	0.13892	0.77516	0.57979	0.14070	0.77602	0.58369	0.15016	0.78038	0.58936	0.15709	0.78481	0.63154	0.18171	0.81603
	0.4	0.57138	0.13681	0.64220	0.57054	0.13829	0.64269	0.57308	0.14033	0.64557	0.57674	0.15694	0.64843	0.60488	0.18169	0.66967
	0.5	0.50000	0.12977	0.50000	0.50000	0.12170	0.50000	0.50000	0.13206	0.50000	0.50000	0.13880	0.50000	0.50000	0.16405	0.50000
	0.6	0.38862	0.09406	0.35780	0.39046	0.09561	0.39731	0.39692	0.10377	0.39443	0.38386	0.10836	0.39139	0.35312	0.13303	0.38033
	0.7	0.25903	0.06376	0.23584	0.26021	0.06468	0.22798	0.25631	0.06978	0.23862	0.25044	0.07229	0.23529	0.20845	0.08709	0.18339
	0.8	0.13409	0.03383	0.11188	0.13334	0.03417	0.11068	0.13187	0.03650	0.10726	0.12832	0.03729	0.10398	0.09876	0.02984	0.07610
	0.9	0.03952	0.01005	0.03337	0.03901	0.01008	0.03096	0.03753	0.01058	0.02977	0.03569	0.01048	0.02764	0.02995	0.00704	0.01671
0.30	0.1	0.31979	0.07307	0.69092	0.31470	0.07483	0.69268	0.32810	0.08014	0.69691	0.32843	0.08093	0.69948	0.34774	0.09197	0.70599
	0.2	0.49868	0.11803	0.59017	0.49989	0.11939	0.59169	0.49864	0.12772	0.59446	0.50019	0.13534	0.59304	0.55820	0.15814	0.59648
	0.3	0.56580	0.13084	0.77637	0.56593	0.13760	0.77801	0.57311	0.15017	0.77113	0.58549	0.16084	0.78115	0.64279	0.19004	0.82188
	0.4	0.56227	0.12940	0.63345	0.56152	0.13783	0.63537	0.56642	0.15956	0.63971	0.57475	0.16339	0.64643	0.61240	0.18763	0.67553
	0.5	0.50000	0.10722	0.50000	0.50000	0.11491	0.50000	0.50000	0.13084	0.50000	0.50000	0.13668	0.50000	0.50000	0.15793	0.50000
	0.6	0.39773	0.08898	0.36455	0.39648	0.09367	0.36583	0.39358	0.11126	0.36029	0.38525	0.11697	0.35397	0.34710	0.13231	0.32447
	0.7	0.27620	0.06448	0.24363	0.27507	0.06808	0.24399	0.26689	0.09009	0.23887	0.25651	0.07970	0.21895	0.19721	0.06316	0.17512
	0.8	0.14732	0.03987	0.12983	0.14611	0.03967	0.12731	0.14136	0.04324	0.11954	0.13081	0.04230	0.11666	0.08280	0.02766	0.08692
	0.9	0.04425	0.01237	0.03950	0.04550	0.01273	0.03762	0.04390	0.01324	0.03509	0.03753	0.01245	0.03252	0.01726	0.00599	0.01401
0.45	0.1	0.30828	0.06904	0.94776	0.30803	0.06976	0.95143	0.31300	0.07811	0.96217	0.31833	0.08145	0.96684	0.34235	0.09181	0.98545
	0.2	0.47519	0.08830	0.84173	0.47685	0.10036	0.85394	0.48729	0.12366	0.87385	0.50015	0.13239	0.88901	0.57779	0.15829	0.92988
	0.3	0.54600	0.09693	0.72440	0.54797	0.11237	0.73749	0.56047	0.14894	0.77943	0.57779	0.15876	0.77244	0.64315	0.19103	0.83674
	0.4	0.54870	0.09553	0.61035	0.54880	0.11409	0.61836	0.55832	0.15198	0.63221	0.57336	0.16479	0.64299	0.61396	0.18873	0.67448
	0.5	0.50000	0.09263	0.50000	0.50000	0.10881	0.50000	0.50000	0.14234	0.50000	0.50000	0.15090	0.50000	0.50000	0.17962	0.50000
	0.6	0.41130	0.08519	0.38965	0.41320	0.09567	0.39344	0.40168	0.13979	0.36779	0.38966	0.12391	0.35911	0.34804	0.13429	0.32954
	0.7	0.29600	0.07053	0.27560	0.29603	0.07499	0.26251	0.27993	0.08932	0.24077	0.26427	0.08032	0.22756	0.19685	0.06831	0.17526
	0.8	0.14681	0.04658	0.15827	0.14615	0.04662	0.14606	0.15281	0.04980	0.13615	0.13985	0.04739	0.13496	0.08221	0.02841	0.07912
	0.9	0.05172	0.01728	0.05224	0.05197	0.01634	0.04657	0.04780	0.01905	0.03780	0.04167	0.01662	0.03516	0.01985	0.00637	0.01453
1.00	0.1	0.28562	0.03898	0.90423	0.28857	0.05260	0.92180	0.29843	0.07093	0.94700	0.30990	0.07968	0.95470	0.34272	0.09174	0.98510
	0.2	0.42383	0.02861	0.75166	0.43258	0.06384	0.78742	0.45476	0.10715	0.83808	0.46804	0.12705	0.85368	0.54183	0.15202	0.91613
	0.3	0.48998	0.02264	0.63337	0.50099	0.06958	0.66925	0.52930	0.13030	0.73089	0.54040	0.14438	0.73815	0.62953	0.18603	0.81152
	0.4	0.51462	0.03185	0.55665	0.52099	0.08153	0.57736	0.53615	0.14398	0.60823	0.54808	0.15707	0.60987	0.60991	0.18843	0.66836
	0.5	0.50000	0.05182	0.50000	0.50000	0.09930	0.50000	0.50000	0.15687	0.50000	0.50000	0.15499	0.50000	0.50000	0.16279	0.50000
	0.6	0.44558	0.07521	0.44339	0.43931	0.10127	0.42064	0.42395	0.15776	0.39177	0.41242	0.13483	0.37933	0.35670	0.13707	0.33164
	0.7	0.35002	0.08028	0.36663	0.33801	0.08034	0.33075	0.31490	0.10622	0.27911	0.29960	0.10004	0.26185	0.21147	0.07451	0.18868
	0.8	0.21819	0.07735	0.24854	0.20962	0.07528	0.21298	0.19524	0.06907	0.16292	0.17196	0.06477	0.14632	0.09819	0.03982	0.08399
	0.9	0.07158	0.03476	0.09977	0.07143	0.03088	0.07820	0.06677	0.02983	0.05309	0.05830	0.02898	0.04530	0.01829	0.00844	0.01490

*Baron, Frank and Michalos, James P., "Laterally Loaded Plane Structures and Structures Curved in Space," *Proceedings*, ASCE, V. 76, Separate No. 51, Jan. 1951.

of the analogy and then evaluated. This involved rather laborious algebra and integration, but it made possible a general tabulation which permitted computation of influence values for any ratio of rise to span, h/L, and of bending to torsional stiffness, K_n/K_t, where $K_n = EI_1$ and $K_t = GJ$. In these expressions, E has its usual meaning, I_1 is the moment of inertia of a cross section about the section's principal axis lying in the plane of the arch, and G is the modulus of elasticity in shear. The term J is a torsion factor such that GJ is the moment about the tangential axis necessary to deform a segment a unit angle per unit length about the same axis. The influence ordinates tabulated are for moments and shear at the support. These moments are about rectangular axes, x and y, lying in the plane of the structure, and are called m_x and m_y, respectively. The shear is normal to the plane of the structure. The sign convention used is illustrated in Fig. 1a. In this figure the moments and shear are represented as vectors. They are considered positive on a front face when they are in the direction of an axis. The moments and shear are considered positive on a back face when they are against the direction of an axis. In Table 1, observe that the moment about the x axis at A, m_{xA}, is negative.

Referring again to Table 1, to obtain the shear v_A at end A for load P, multiply by P the tabulated value which corresponds to the position of the load. If more than one load is on the arch, repeat the above step for each load and add the separate values obtained. The procedure for the x component of moment at end A, m_{xA}, and the y component of moment, m_{yA}, is the same except that for m_{xA} multiply the tabulated values by Ph, and for m_{yA} multiply them by PL. Although the tabulated influence ordinates are for moments and shear at a support only, by means of these values and computations of statics, influence values may be conveniently determined for any section of the arch. In addition, by means of the relationships shown in Fig. 1b, normal and tangential components of moment may be readily computed.


Y
Z
X
The structure lies in the XY–plane and is loaded normal to this plane.
Y
X
Back Face
Front Face
⊕ Conotes in the direction of the Z-axis
⊙ Conotes opposite to the direction of the Z-axis
(a) Positive Values of Moments and Shears Acting on a Segment of the Structure
Y
X
(b) Relationships for Bending and Torsional Moments

Note: Use the right hand rule for positive sense of moment.

Fig. 1—Sign convention and moment relationships

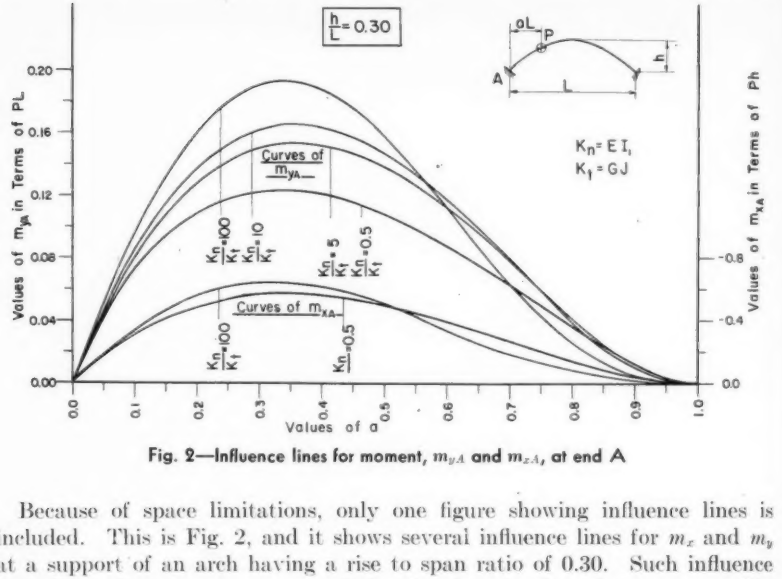


Fig. 2—Influence lines for moment, m_{yA} and m_{xA}, at end A

Because of space limitations, only one figure showing influence lines is included. This is Fig. 2, and it shows several influence lines for m_x and m_y at a support of an arch having a rise to span ratio of 0.30. Such influence lines have the advantage over tabulated values in that they more readily give the user a sense of scale. In addition, they enable one to interpolate quickly and accurately. Influence lines or a study of Table 1 will indicate how relatively insensitive m_{xA} is for different ratios of h/L and K_n/K_t. This is also true for v_A. It is not true for m_{yA}.

A procedure for drawing curves of moments

To study the variation in moments along a structure resulting from some pattern of loading, separate curves are drawn for moments about x axes and for moments about y axes. This procedure lends itself to quick sketching of approximate moment diagrams and to the development of judgment. In addition, exact moment curves may be conveniently drawn by this procedure with the aid of influence values.

Consider the single arch rib of Fig. 3a. This arch is subjected to a load P normal to its plane and applied at the crown of the arch. Due to the symmetry of the structure and of the load, the x components of moment, m_x, can be determined by a consideration of statics alone. The moment diagram showing the distribution of the y components of moment, m_y, is drawn in two steps. First draw the curve of moments which would exist if the structure were a simply supported beam of span L. This moment diagram satisfies the requirements of statics. To complete the diagram showing the distribution of m_y, draw a closing line such that the requirements of geometry are also satisfied. Suppose that the arch had a zero rise, *i.e.*, it were a fixed-end beam. If the beam had a constant cross section, the end moments would

be $\frac{1}{8}$ PL and the closing line would be as shown dashed on the figure. Of course, if the beam were not of constant cross section, the position of the closing line would still have to be such as to balance the angle changes along the beam. Now consider again the arch of rise h. In this case it is necessary to apply a correction to the curve of moments of the fixed-end beam. This correction adjusts for the effect of the rise on the distribution of moments.

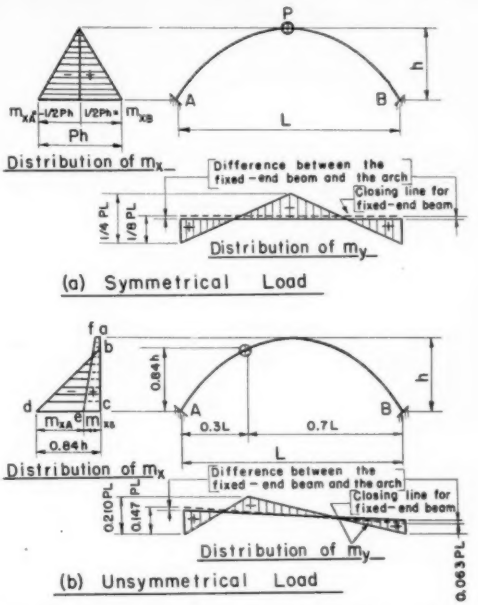


Fig. 3—Moment diagrams for concentrated loads normal to the plane of an arch

In Fig. 3b the loading is not symmetrical, and consequently the exact distribution of m_x cannot be determined by statics alone. The curve of moments for m_x is drawn in two steps. First draw a curve which satisfies the requirements of statics. The moment curve *abcdb* is such a curve. Now make an estimate of the distribution of this moment to the two springings. For example, *de* represents m_x at A, and *cc* represents m_x at B. Next, make an estimate of the moment at the crown. For example, *fa* represents this moment. Finally, draw line *ef*. The distribution of m_x between the crown and A is represented by *abdef*, whereas the distribution between the crown and B is represented by *acef*. The distribution of m_y is also obtained in two steps, as outlined previously. The diagrams showing the distribution of m_x and m_y are correct, regardless of the shape of the structure, if the angle changes balance. "Exact" moment diagrams for parabolic arches are discussed later.

It is pertinent at this time to point out that three stiffness factors (K_{xx}, K_{yy} and K_{xy}) are involved in the procedure* used to determine the influence values of Table 1. These stiffness factors, written in terms of the bending and torsional stiffnesses, relate moments and angle changes about x and y axes. Because of the curvature of the arch, the values of the stiffness factors vary continuously along the arch axis. Furthermore, angle changes about either an x or y axis depend upon the moments about both axes. These

*Ibid.

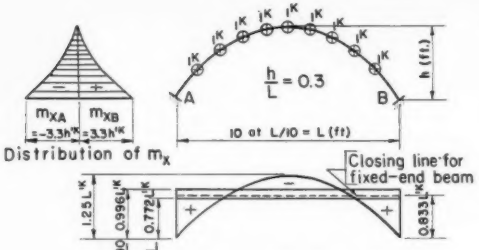


Fig. 4—Distribution of moments in arches of different stiffness ratios

statements must be remembered if it is desired to make any computations of slopes or deflections of the arch.

For parabolic arches, the influence values may be used to draw exact moment curves. For example, to obtain the curve of moments m_x in Fig. 3b, first lay off the statical moment as before. Then, by means of the influence values, determine m_x and v at A. Having these values, determine m_x at the crown by statics. Now draw the closing line, *fe*, and the resulting distribution of moments can be read directly. To obtain the curve of moments m_y, first draw the moment curve for a simply supported beam, as shown in Fig. 3b. From the influence values determine m_y and v at A, then by statics compute m_y at B. Now draw the closing line and obtain the distribution of moments.

Numerical studies, including the effect of haunching

In Fig. 4 is shown a parabolic arch whose rise to span ratio is 0.3. Lateral forces of one kip are applied at each one-tenth point of the span. Moment diagrams are included for two ratios of bending to torsional stiffness: K_n/K_t equal to one, and K_n/K_t equal to 100. Because of the symmetrical loads, the distribution of m_x is the same for both ratios, and can be determined directly from a consideration of statics. Of course, the same values can be obtained by means of the influence ordinates. The m_y diagram was drawn in two steps, using the influence values in obtaining the moments at the supports. A solid closing line is shown for each ratio of K_n/K_t considered. The dashed closing line is for a fixed-end beam. For convenience, this line was located by treating the polygonal diagram as a parabola, which it very nearly is. The figure indicates that a good estimate of the m_y diagram can be made from a consideration of the moment diagram for a fixed-end beam. The two solid lines bracket the position of the closing line for all values of K_n/K_t between one and 100.

Attention is called to the fact that influence values can be determined for arches having particular variations along their length of the ratio of bending to torsional stiffness. But many variations of this ratio are possible. A reinforced concrete arch may have a variation in a particular case from, say, 1.0 at the crown to 0.6 at the springing. In another case the variation may be from, say 100 at the crown to 25 at the springing. Several numerical studies were made, by means of the shear and torsion analogy,* to get a

*Ibid.

scale on the effect of haunching. The following cases were considered:

1. h/L equal to 0.15, and K_n/K_t varying from 1.0 at the crown to 0.60 at the springing. Such a variation is quite probable in a reinforced concrete arch rib.
2. h/L equal to 0.45, and K_n/K_t varying as in case 1.
3. h/L equal to 0.15, and K_n/K_t varying from 100 at the crown to 25 at the springing. Such a variation is quite probable in a barrel arch.

Table 2 contains values of moments and shears, at a support of each of the above arches, resulting from a unit load placed at a quarter point or at the crown. If these values are plotted in each case alongside the influence line corresponding to the biggest K_n/K_t ratio of the particular arch considered, it will be noted that the values of m_x "hug" the influence line and that the values of m_y are increased relatively little by haunching. This indicates how useful the influence values for constant cross section may be even in cases of wide variation in cross section such as are fairly common in reinforced concrete design.

Fig. 5 contains moment curves for the three arches of Table 2. In this figure the arches are subjected to unit loads applied at the crown and at the quarter points. On each diagram showing the distribution of m_y, two closing lines have been drawn. The solid line is the correct one for the actual variation in the ratio K_n/K_t. The dashed line is for a constant value of this ratio. The value used in drawing this line is the largest K_n/K_t ratio of the arch under consideration. As a rough average, it may be said that the haunching increases m_y at the supports by, say, 10 percent. The fixed-end moment for a beam of constant cross section would be $0.313L$ in each case. For a haunched beam this value would be somewhat higher. Thus, good approximations for the distribution of moments in an arch of varying cross section may be obtained by treating the arch as if it had a constant cross section and

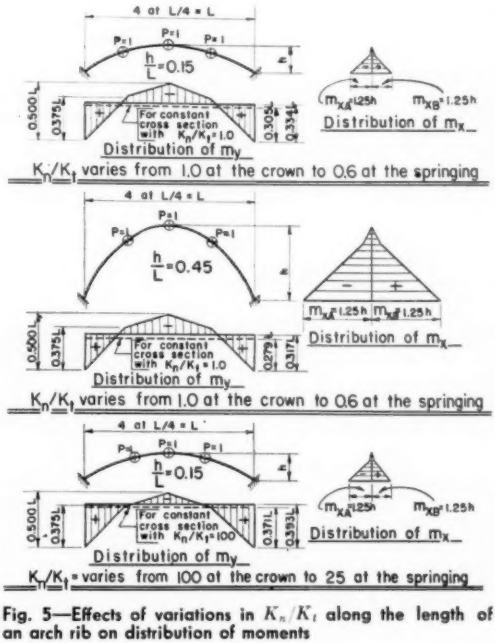
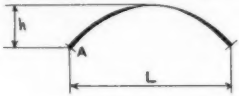


Fig. 5—Effects of variations in K_n/K_t along the length of an arch rib on distribution of moments

TABLE 2—EFFECTS OF HAUNCHING ON MOMENTS AND SHEARS



Values of K_n (relative)	Values of K_n/K_t	Values of h/L	Distance of Unit Load from A	Values of m_{xA} (−)	Values of m_{yA} (+)	Values of v_A (+)
Varies from 1.00 at crown to 1.76 at springing	Varies from 1.00 at crown to 0.60 at springing	0.15	0.25 L	0.586 h	0.150 L	0.852
			0.50 L	0.500 h	0.136 L	0.500
			0.75 L	0.164 h	0.048 L	0.148
Varies from 1.00 at crown to 1.76 at springing	Varies from 1.00 at crown to 0.60 at springing	0.45	0.25 L	0.540 h	0.124 L	0.806
			0.50 L	0.500 h	0.125 L	0.500
			0.75 L	0.210 h	0.068 L	0.194
Varies from 1.00 at crown to 2.00 at springing	Varies from 100 at crown to 25 at springing (barrel arch)	0.15	0.25 L	0.611 h	0.177 L	0.877
			0.50 L	0.500 h	0.166 L	0.500
			0.75 L	0.139 h	0.050 L	0.123

a ratio of bending to torsional stiffness equal to that at the crown. Fairly good approximations also may be obtained by a consideration of results for fixed-end beams.

RIBS BRACED WITH VERY FLEXIBLE OR VERY STIFF STRUTS

This section consists of a study of the influence of bracing on the distribution of moments and shears in arches subjected to lateral loads. Two extreme cases of bracing are considered. These are arch ribs braced with very flexible struts and arch ribs braced with very stiff struts. No attempt is made to study the effects of struts having intermediate stiffnesses, but rather this section is concerned with the designer's interest in the influence of bracing and the possible magnitude of this influence on moments and shears in arches.

Procedure for drawing curves of moments

The effects of very flexible struts can be studied by means of the influence values obtained for unbraced arches and by a consideration of results for fixed-end beams. These struts are considered to offer no resistance to bending or torsion in the rib. However, they are considered to be inextensible and to distribute the load equally between the ribs. Thus, the results for two ribs braced with very flexible struts are one-half of those obtained for the unbraced arch. The effects of very stiff struts on moments and shears in the arch can be studied with the aid of an analytical procedure* developed for arches with rigid struts, and by comparison with an open-web girder.

Fig. 6 illustrates the method used in drawing curves of moments for arches braced with struts. The arches and loads are symmetrical. In (a) the values for the arch with very flexible bracing are one-half those obtained for

*Michalos, James P., "Analytical Procedures for Structures in Space and a Study of Wind on Arches." See bottom footnote, p. 377.

an unbraced rib. In (b), which considers the arch with very stiff bracing, the diagram showing the distribution of m_y is again drawn in two parts. Because of the rise of the arch, the closing lines are not horizontal as in an open-web girder with very stiff struts, but are tilted and shifted somewhat. The diagram showing the distribution of m_x is now also obtained in two steps. First draw the curve of moments as if the struts were very flexible. Then draw closing lines through points on the diagram approximately corresponding to the mid-points of the panels. If the arch had all rise and no span, *i.e.*, if it were a cantilevered open-web girder with very stiff struts, the closing lines would be vertical. However, because the structure has both rise and span, the closing lines are tilted and shifted.

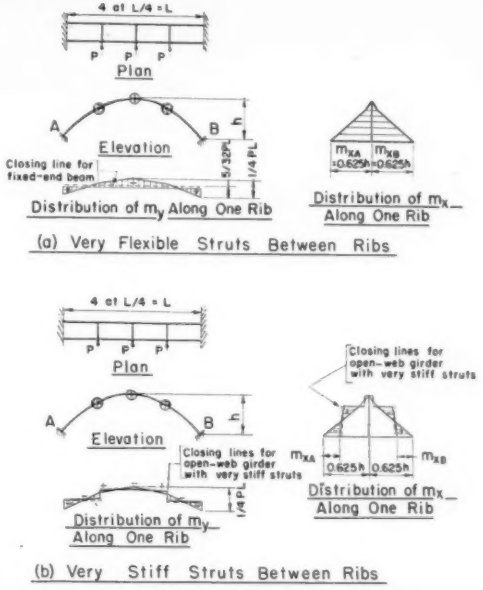


Fig. 6—Procedure for drawing curves of moments

This procedure for drawing curves of moments is useful for qualitative and quantitative studies. In the numerical studies which follow, scale is obtained on the amount of tilting and shifting which may be involved in "correcting" the moment curves of the open-web girder.

Numerical studies

By means of the analytical procedure* for arches braced with rigid struts, four parabolic arches were studied to determine the effect of very stiff struts. The resulting moments and shears are compared with those which would occur if the arches were braced with very flexible struts. The results for the arches considered braced with very flexible struts were determined by using influence values. In the analysis of the arches with very stiff struts, the ribs were considered to be segmental between adjacent struts.

Table 3 is a summary of computed shears and moments in the arches braced with very stiff struts, and with very flexible struts. This table shows the shear, bending moment, m_n, and torsional moment, m_t, at a springing, at the crown and at two intermediate sections for different positions of a

Ibid.

TABLE 3—SUMMARY OF COMPUTED SHEARS AND MOMENTS IN ARCHES BRACED WITH VERY STIFF STRUTS AND WITH VERY FLEXIBLE STRUTS. Different ratios of rise to span and bending to torsional stiffness are considered.

Case	Unit Load at	Shears and Moments at End A			Shears and Moments at Station 2			Shears and Moments at Station 4			Shears and Moments at Station 6		
		V	M_x	M_y	V	M_x	M_y	V	M_x	M_y	V	M_x	M_y
I $\frac{f}{L}=0.15$ Section $K_x=486\times10^7$ lb-ft^2 $K_{xt}=350\times10^7$ lb-ft^2 $K_y=486\times10^7$ lb-ft^2	2	* 0.422 ** 0.455	9.490 25.692	-0.311 -0.030	0.422 0.455	-9.462 -4.792	-0.311 -0.839	-0.078 -0.045	1.617 -2.983	-0.203 -0.130	-0.078 -0.045	1.574 -1.160	0.203 0.280
	4	* 0.338 ** 0.360	7.669 29.983	-0.149 -0.184	0.338 0.360	-7.547 3.634	-0.327 -2.028	0.338 0.360	-7.120 -11.743	-0.327 -1.248	-0.162 -0.140	3.227 -4.160	0.177 0.520
	6	* ±0.250 ** ±0.250	5.698 17.184	0.120 -0.184	±0.250 ±0.250	-5.640 -3.793	-0.338 -1.993	±0.250 ±0.250	-5.224 -5.080	-0.338 -2.033	±0.250 ±0.250	-5.082 -13.360	±0.278 0
II $\frac{f}{L}=0.45$ $K_x=486\times10^7$ lb-ft^2 $K_{xt}=350\times10^7$ lb-ft^2 $K_y=486\times10^7$ lb-ft^2	2	* 0.407 ** 0.445	14.797 25.699	-1.076 -1.624	0.407 0.445	-14.731 -7.000	-1.076 -2.966	-0.092 -0.093	2.436 -4.682	0.799 -0.873	-0.092 -0.093	1.923 -2.360	0.787 0.840
	4	* 0.331 ** 0.345	12.095 32.842	-0.349 -2.903	0.331 0.345	-11.896 7.033	-0.919 -6.064	0.331 0.345	-8.963 -13.937	-0.919 -4.073	-0.169 -0.195	3.509 -7.720	1.119 1.290
	6	* ±0.250 ** ±0.250	9.187 29.896	0.644 -1.781	±0.250 ±0.250	-8.935 11.114	-0.967 -5.407	±0.250 ±0.250	-6.719 -4.493	-0.967 -6.564	±0.250 ±0.250	-5.272 -17.040	±0.854 0
III $\frac{f}{L}=0.45$ $K_x=36\times10^7$ lb-ft^2 $K_{xt}=518\times10^7$ lb-ft^2 $K_y=0.6\times10^7$ lb-ft^2	2	* 0.388 ** 0.470	13.299 31.295	-0.009 0.737	0.388 0.470	-13.301 -2.816	-0.009 -1.508	-0.131 -0.030	3.930 -1.970	0.006 -0.788	-0.131 -0.030	3.742 -0.580	0.006 -0.840
	4	* 0.300 ** 0.385	10.811 40.520	-0.004 2.886	0.300 0.385	-10.794 12.742	-0.004 -1.788	0.300 0.385	-8.065 -8.886	0.005 -1.542	-0.207 -0.119	4.185 -4.080	0.005 -1.260
	6	* ±0.250 ** ±0.250	9.034 32.977	0.004 3.044	±0.250 ±0.250	-9.005 14.647	-0.009 -1.186	±0.250 ±0.250	-6.726 0.240	-0.005 -3.723	±0.250 ±0.250	-5.226 -13.320	±0.004 0
IV $\frac{f}{L}=0.45$ $K_x=486\times10^7$ lb-ft^2 $K_{xt}=350\times10^7$ lb-ft^2 $K_y=486\times10^7$ lb-ft^2	2	* 0.425 ** 0.445	8.312 25.699	-0.257 -1.624	0.425 0.445	-7.184 -7.000	-0.306 -2.966	-0.055 -0.055	0.924 -4.682	0.154 -0.873	-0.055 -0.055	0.725 -2.360	0.190 0.840
	4	* 0.342 ** 0.345	6.772 32.842	0.060 -2.903	0.342 0.345	-5.709 7.033	-0.371 -6.064	0.342 0.345	-4.337 -13.937	-0.238 -4.073	-0.158 -0.195	1.573 -7.720	0.305 1.290
	6	* ±0.280 ** ±0.250	4.993 29.896	0.603 -1.781	±0.250 ±0.250	-4.124 11.114	-0.332 -5.407	±0.250 ±0.250	-2.123 -4.493	-0.391 -6.564	±0.250 ±0.250	-2.588 -17.040	±0.136 0

* Shears and moments in an arch braced with very stiff struts. Values are for a single rib.

** Shears and moments in an arch braced with very flexible struts.

unit lateral load. All arches considered are 240 ft long, with ribs spaced 24 ft center to center. Two ratios of rise to span, 0.15 and 0.45, and two patterns of strut spacing are included. Three of the arches have ribs with rectangular cross sections and one arch has an I section, more common to steel construction. For each position of the unit load on the very stiffly braced arch, there are in general two values for each tabulated function, the values differing on either side of the intersection of a strut with a rib. Only the larger value is shown. This is the upper figure of each pair listed for the different positions of the unit load. The lower figure represents the value for the ribs connected by very flexible struts. Attention is called to the fact that in the arch with very stiff struts moments also occur about axes 2-2. However, these are negligible compared to the moments about such axes resulting from gravity loads and have not been included.

Fig. 7 contains moment curves for the arch of Case II in Table 3. The arch is symmetrically loaded with concentrations of one kip. Similar curves may be drawn for the arch unsymmetrically loaded. The figure shows the distribution of m_x and m_y considering the arch braced with very flexible struts or with very stiff struts. Observe the relatively small tilting and shifting of the closing lines for the arch with very stiff struts.

A study of Table 3 and Fig. 7 indicates that (a) stiff bracing greatly reduces the large bending moments at the springings and at the crown; (b) such bracing in general reduces the torsional moments along the arch; and (c) stiff bracing may increase some moments.

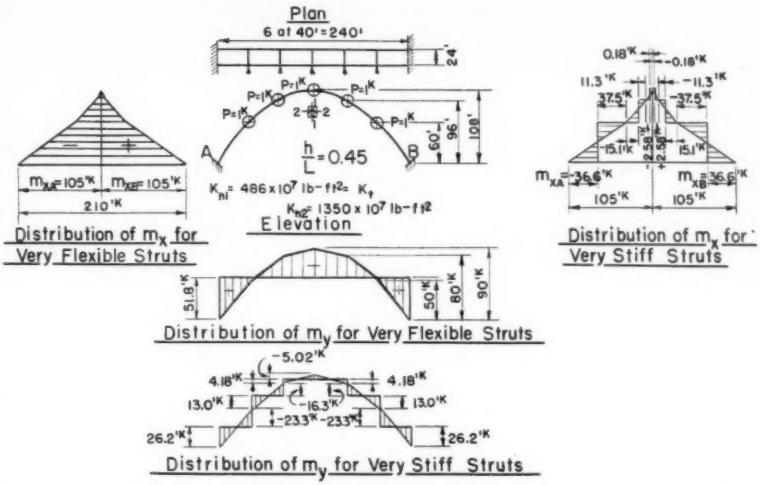


Fig. 7—Comparison of moments along a rib of an arch with very flexible struts and with very stiff struts. A symmetrical load

CONCLUDING REMARKS

Qualitative and quantitative studies have been made of moments and shears resulting from lateral loads applied to arch ribs. Curves for the x and y components of moment are drawn in two steps, first drawing a curve which satisfies statics and then making a planar correction so as to satisfy geometry also. Influence values and results obtained from a consideration of fixed-end beams and open-web girders are used to obtain exact and approximate curves of moment.

Title No. 47-30

An economical system of framing for concrete slabs is offered for further development by the profession

Getting More for Our Concrete Dollar*

By I. E. MORRIS†

SYNOPSIS

A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. It is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. Members are designed to make maximum use of both concrete and steel.

The design of solid concrete slabs normally involves a considerable waste of material, principally because of the assumption that concrete has no tensile value. For instance, in the case of a slab with an effective depth of 6 in., when $k = 0.403$, the distance from the top fibers to the neutral axis is only 2.42 in., leaving 3.58 in. of concrete which performs no useful purpose. The concrete below the steel protects it against fire or the elements.

For a simple span, of say 20 ft, the usual procedure, in fireproof construction, is to use beams and solid slabs, or a ribbed slab. The use of beams involves expensive formwork; the ribbed slab is reasonably economical, and is satisfactory where an exposed ceiling is not desired. But even in a ribbed slab the maximum effectiveness of the concrete is seldom obtained, due to the fact that, if the effective depth is such as to develop the full compressive resistance of the concrete above the neutral axis, the required steel area becomes excessive, or the shear in the joists requires that the ends be tapered, or deflection becomes troublesome, or perhaps there is a combination of all these difficulties. The problem, therefore, is to devise a member which will utilize both concrete and steel to maximum advantage and at the same time have sufficient effective depth to span a reasonable distance without excessive deflection. This means that all structural concrete in a member should be put to work. Filler material should be light and inexpensive.

The system of framing presented here is intended primarily for roofs where an exposed ceiling is desired for economy. No attempt has been made to make of the system a classroom problem illustrating the finer points

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of theory, or an exercise in higher mathematics. The controlling idea is to show how a simple application of fundamental principles can produce greater economy by using concrete more effectively. It is hoped that other engineers will be interested in further developing the idea.

Fig. 1 shows a concrete slab with alternating horizontal elements connected by sloping elements. The topmost areas, shown cross-hatched, are used to resist compressive stresses, thus taking advantage of an economical depth and a more effective use of the compressive qualities of concrete. The slope of the inclined portions is held to a value which will permit placing the concrete without trouble from the force of gravity. The thickness t_1 is made equal to kd, so that the neutral axis is always at the bottom of the upper horizontal elements; and an area of steel is calculated to balance the compressive resisting moment, which is the value of the rectangular portion $t_1 \times 2a$ plus the two small triangles. Tensile steel is placed in the lower horizontal portions as shown. Welded wire mesh is placed across the entire member to resist temperature stresses and transverse bending.

This system of framing may also be used for continuous spans, in which case, at the supports, tensile steel is placed in the upper horizontal sections.

It should be noted that the inclined portions of the member produce a horizontal thrust against the horizontal elements. For interior members of a panel; that is, where there are several units of width b, the thrusts counteract each other, but the lower horizontal portion of the member at the beginning of a series must be designed to resist the thrust. This is done by considering the section $b/2 + a$, see Fig. 6, as a narrow deep beam, with a load per foot equal to the thrust per foot of span. Steel to resist the resulting moment is placed near the outer edge.

The width, b, of members may be varied to fit physical requirements of the structure by making proper substitutions in the equations.

If the framing is a roof over classrooms, for instance, it may be desirable to have some of the lower horizontal elements centered over partitions between the rooms. This may easily be done by the variations mentioned above. This type of construction may also be used for floors by filling between the inclined elements with a lightweight concrete, or any other material having requisite compressive strength to support partitions and concentrated loads. Even for roofs, the drainage problem may require a fill between the inclined elements, but in such instances the structural concrete below the neutral axis is held to a minimum, and filler material is light and inexpensive.

The system is flexible and can be made to span considerable distances, either as a simple span or as a series of continuous spans. It is realized that the following analysis involves certain assumptions which are not exact, but they are on the safe side. In reinforced concrete design, few assumptions, if any, are exact. The individual designer, however, is at liberty to extend the analysis to any desired degree of mathematical precision.

To prevent excessive deflection, it is recommended that the minimum effective depth-to-span ratio be 1/30 for floors and 1/35 for roofs.

RESISTING MOMENTS and STEEL AREAS

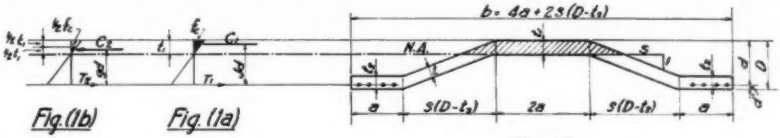


Fig.(1b) Fig.(1a)

Fig.(1)

$t_1 = kd$, where $k = \frac{nf_c}{nf_c + f_s}$

$Jd = d - t_1/3$

$gd = d - 2t_1/3$

$M_1 = \frac{1}{2} f_c \cdot t_1 \cdot 2a \cdot Jd$

$M_2 = \frac{1}{2} f_{c/2} \cdot st_1 \cdot t_1 \cdot gd$

$A_{s_1} = \frac{M_1}{f_s\, Jd} = \frac{f_c}{f_s} \cdot at_1$

$A_{s_2} = \frac{M_2}{f_s\, gd} = \frac{1}{3} \frac{f_c}{f_s} \cdot st_1^2$

$\therefore$ Resisting Moment $M = M_1 + M_2 = f_c t_1 (ajd + st_{1/4} \cdot gd)$ ——— (1)

$\therefore$ Area of steel $A_s = A_{s_1} + A_{s_2} = \frac{f_c}{f_s} \cdot t_1 (a + st_{1/3})$ ——— (2)

Notes:

*(1) Because the edges of the upper element are likely to be rounded in construction, this expression used instead of the more accurate one, which is: $M_2 = st_1^2 \cdot f_{\frac{c}{2}} (d - \frac{t_1}{3})$.

(2) The thickness, t_1 of the sloping element may normally be equal to t_2.

SHEAR

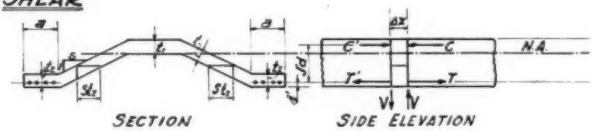


Fig.(2) Fig.(2a)

Total horizontal shearing force acting on a plane between the neutral axis and the steel is T'-T. The area of this horizontal plane is $2st_2(\Delta x)$.

$\therefore$ Unit shearing stress $V = \frac{T' - T}{2st_2(\Delta x)}$ ——— (a)

The element considered is in equilibrum under forces shown, that is,

$V(\Delta x) = (T'-T)\, J'd$ or $T'-T = \frac{V(\Delta x)}{J'd}$ ——— (b)

Substituting (b) into (a), we obtain,

$v = \frac{V}{2st_2 J'd}$ ——— (3)

where J'd is a value between .Jd and gd, and equals .96 Jd. approximately.

If $a + sd' < st_2$, the maximum unit shearing stress will occur at a plane immediately above the steel. Then,

$v = \frac{V}{2(a + sd')J'd}$ ——— (3A)

A more exact form of Equation (3) is,

$v = \frac{V}{2\left(\frac{st_2 + 1}{\sqrt{st_2 + 1}}\right) t \cdot J'd}$ ——— (3B)

HORIZONTAL THRUST

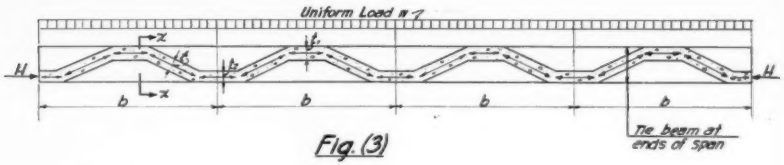


Fig. (3)

Treating each section as a symmetrical hip frame, with uniform vertical load, we have the condition shown in Fig. (4).

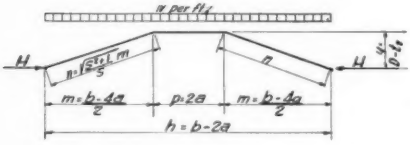


Fig. (4)

These bars to resist temperature stresses and reaction due to horizontal thrust.

Stirrups to support bars where beam bears on wall, as required for shear otherwise.

SECTION X-X

$$H = \frac{w}{4} \times \frac{n \cdot m(4h-3m) + p[p^2 + 6m(h-m)]}{4(2n+3p)} \qquad (4)$$

Formula (4) assumes a hinged condition at each end of the member. This assumption is not exact, but it is considered sufficiently so since the restraint is slight.
Since a high degree of accuracy is unnecessary, Formula (4A), which is much simpler, may be used for calculating the horizontal thrust Thus,

$$H = \frac{wh^2}{64} \qquad (4A)$$

The lower portion of the outermost section must be designed to resist the thrust H. See Figure (5)

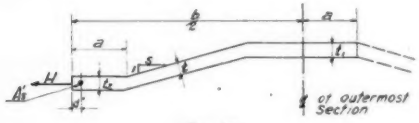


Fig. (5)

Since the extreme outer edge of the element shown in Fig. (5) is in tension, the width $b_2 + a$ becomes the depth of a horizontal beam which produces additional compression in the upper horizontal portion. Treating H as a load per foot, with a span of L, the moment is

$$M' = \frac{HL^2}{8} \qquad (5)$$

The required steel area is

$$A'_s = \frac{M'}{f_s\left(\frac{7}{8}\right)\left(\frac{b_2}{2} + a - d'\right)} \qquad (6)$$

COMPRESSIVE STRESSES IN THE TOP OUTERMOST HORIZONTAL PORTION

The outermost top horizontal portion is stressed by three forces:
(a) Flexural Stress, f_{c1}, resulting from vertical loads;
(b) Direct stress, f_{c2}, due to thrust of inclined elements against the horizontal elements;
(c) Flexural Stress, f_{c3}, resulting from the outermost section of width $\frac{b_2}{2}+a$ acting as a horizontal beam.

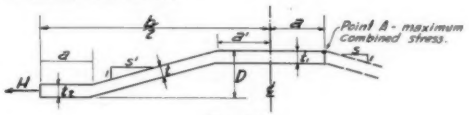


Fig. (6)

Since, in Fig. (6), $s' = \frac{\frac{b_2}{2}-a-a'}{D-t_2}$ is limited to 4 (this is merely a suggestion)

$$a'_{max.} = \frac{b_2}{2} - a - 4(D-t_2) \qquad (7)$$

The stress f_{c1} is given by Equation (1) when $a'+a = 2a$, or when $a'=a$. But, when a' is increased in order to keep the combined stresses at point A, Fig. (6), within the allowable, Formula (1A) should be used.

$$f_{c1} = \frac{M}{t_1\left(\frac{a'+a}{2}jd + \frac{st_1+s't_1}{8}gd\right)} \qquad (1A)$$

$$f_{c2} = \frac{H}{12t_1} \qquad (8)$$

f_{c3} is given by Formula (9), the derivation of which follows: see Figures (7) and (7a)

$$x = k\left(\frac{b_2}{2}+a-d'\right) \qquad e = \frac{b_2}{2}+a-d'-\frac{x}{3}$$

$$\because M' = \tfrac{1}{2} f_{c3} \cdot x \cdot t_1 \cdot e, \text{ (when } x \le a+a')$$

$$f_{c3} = \frac{2M'}{x \cdot t_1 \cdot e} \qquad (9)$$

Fig. (7a)

Fig. (7)

The maximum combined stress occurs at Point A, Fig. (7), and is

$$f_{c\,max.} = \sqrt{(f_{c1}+f_{c3})^2 + (f_{c1})^2}$$

Illustrative Example 1:

Find the resisting moment and area of steel required in a section when $D = 10''$, $d' = 1''$, $a = 12''$, $s = 3$, $n = 10$, $f_c = 1350$, $f_s = 20{,}000$

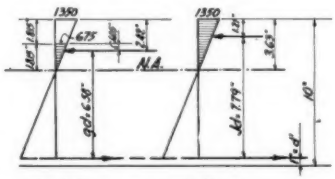


Fig. (8) Fig. (8a)

$$k = \frac{10(1350)}{10(1350)+20{,}000} = .403 \qquad jd = 9 - 1.21 = 7.79''$$

$$t_1 = kd = .403(10-1) = 3.63'' \qquad gd = 9 - 2.42 = 6.58''$$

From Equation 1,

$$M = 1350(3.63)\left(12 \times 7.79 + \frac{5 \times 3.63}{4} \times 6.58\right) = 50.41^{k''}$$

From Equation 2,

$$A_s = \frac{1350}{20{,}000}(3.63)\left(12 + \frac{5 \times 3.63}{4}\right) = 4.06''^2$$

Illustrative Example (2):

Design a slab of shape shown in Fig. (1) to support a total uniform load of 80 p.s.f. (LL+DL) to span 20'-0". Width of each section is 10'-0".
Let $f_c = 1350$, $f_s = 20{,}000$, $n = 10$, $D = 9"$, $d = 8"$, $t = 3"$, $t_1 = 3"$, $a = 1'-0"$, $t_1 = kd$

$$M = \frac{.08 \times 20^2}{8} = 40.0^{k'}$$

From Table I, with $b = 10'0"$, $D = 9"$, and $t_1 = 3.22"$ the resisting moment is shown as $40.5^{k'}$. The steel area required to balance this moment is $3.66\square"$. Therefore, the steel area required for the actual moment is,

$$A_s = \frac{40.0}{40.5} \times 3.66 = 3.62\square", \text{ which may be supplied by } 6\text{-}\tfrac{7}{8}^{\phi}$$

Note: A slab with a D of 10" could also be used, in which case

$$A_s = \frac{40.0}{30.9} \times 4.09 = 3.22\square" \quad \text{Supplied by } 8\text{-}\tfrac{3}{4}^{\phi} \text{ or } 11\text{-}\tfrac{5}{8}^{\phi}$$

SHEAR:

$$V = .08 \times 10 \times \frac{20}{2} = 8.0^{k}$$

$s(D-t_1) = 36"$ $\quad s = \frac{36}{6} = 6$ $\quad a+s = 12+6 = 18"$
$st_1 = 6 \times 3 = 18"$
$jd = 8 - \frac{3.22}{3} = 6.93"$ $\quad j'd = .96(6.93) = 6.65"$

Use Equation (3)

$$v = \frac{8000}{2 \times 18 \times 6.65} = 33.5^{\#/\square"} \quad (60^{\#/\square"} \text{ permitted by A.C.I. Code})$$

Illustrative Example (3):

(a) Find the horizontal thrust per foot on the slab designed in Example 2, using both methods of determining H.
(b) Check moments and shears in the plane perpendicular to the longitudinal axis; that is, moments and shears due to transverse bending.
(c) Design the edge member to resist horizontal thrust.
(d) Design the outermost top horizontal portion for combined stress.

See Figures (9), (a and b), and (10)
Here, $w = .08^{k'}$, $h = 10'0" - 2'-0" = 8'0"$, $y = \frac{9-3}{12} = 0.5'$, $m = \frac{10-4}{2} = 3'0"$, $n = \frac{\sqrt{37}}{6} \times 3' = 3.04'$,
$p = 2 \times 1 = 2'-0"$

(a) Using Equation (4), $H = \frac{.08}{4} \times \frac{3.04(3)(32-9)+2[4+18(8-3)]}{0.5(2 \times 3.04+6)} = 1.32^{k}$ per ft. of span.

Using Equation (4A), $H = \frac{.08 \times 8^2}{8 \times .5} = 1.28^{k}$ per ft. of span.

(b) A uniform load produces moments and shears over the distance (b-2'). The moments and shears are very small, however, and may be resisted by temperature reinforcement, which in this problem is wire mesh. The moment and shear diagrams are shown in Fig. (9), (a) and (b).

Fig. (9)

(c) See Fig.(10)

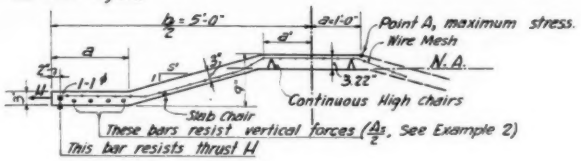


Fig. (10)

Here, $H = 1.32^k$ $L = 20'-0"$
From Equation (5),

$$M' = \frac{HL^2}{8} = \frac{1.32 \times 20^2}{8} = 66.0^{k'}$$

$$A_s' = \frac{M'}{f_s(\frac{7}{8})(\frac{b_2}{2} + a - 2")} = \frac{66 \times 12 \times 1000}{20{,}000 \times \frac{7}{8}(60+12-2)} = .646^{\square"} \quad \text{Use } 1\text{-}1"\phi$$

The reaction on the tie beam at each end of the span is $1.32 \times 10 = 13.2^k$. The Shear value of the concrete is $3 \times 70 \times \frac{7}{8} \times 60 = 11.0^k$, which indicates that $13.2 - 11.0 = 2.2^k$ of the total shear must be resisted by the wire mesh. (In the calculation for the shear value of the concrete, the slight extra thickness of the top horizontal element was disregarded).

(d) Equation (7): $a'_{max.} = 60 - 12 - 4(9-3) = 24"$, and $S' = 4$

Equation (1A): $$f_{c_1} = \frac{40 \times 12 \times 1000}{3.22\left[\frac{24+12}{2} \times 6.93 + \frac{19.32+17.88}{8} \times 5.84\right]} = \frac{480{,}000}{478} = 1005^{\#/\square"}$$

Equation (8): $$f_{c_2} = \frac{1.32 \times 1000}{12 \times 3.22} = 34.2^{\#/\square"}$$

Equation (9): $x = .403(60+12-2) = 28.2"$ $\quad e = 60+12-2-9.4 = 60.6"$

$$f_{c_3} = \frac{2 \times 66 \times 12 \times 1000}{28.2 \times 3.22 \times 60.6} = 288^{\#/\square"}$$

Equation (10): $$f_{c_{max.}} = \sqrt{(1005+288)^2 + 34.2^2} = 1293^{\#/\square"} \quad \text{o.k.}$$

Note: Assuming the maximum stress to be at Point A, Fig.(10) is, of course, a convenient assumption, but it is on the safe side. The sloped portion shown by broken lines could hardly be prevented from resisting some of the compressive stress f_{c_3}, when the span is 20'-0"; however, should additional compression area be desired, the dimension of 3.22", or the distance a', could be increased.

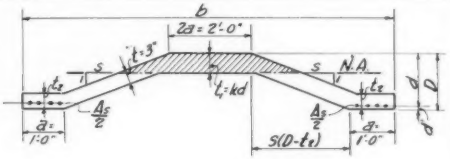
TABLE 1

RESISTING MOMENTS and STEEL AREAS

$a = 1'\text{-}0''$ $t_2 = 3''$ $d' = 1''$ min.

M = Bending Moment, ft-kips on section of slab of width b, in feet.

A_s = Area of steel, sq. ins.



Enter table with known M and width b. Choose a depth, D, having a resisting moment $\geq$ the actual moment. Then, the required steel area is,

$$A_s = \frac{M(\text{actual})}{M(\text{tabulated})} \times A_s(\text{tabulated})$$

f_s	f'_c and n	f_c	t_1 / M / A_s	$b = 8'\text{-}0''$ D 7"	D 8"	D 9"	$b = 10'\text{-}0''$ D 9"	D 10"	D 11"	D 12"	$b = 12'\text{-}0''$ D 11"	D 12"	D 13"	D 14"	D 15"
20,000	2500 12	875	t_1	2.06	2.41	2.76	2.76	3.10	3.44	3.78	3.44	3.78	4.13	4.47	4.81
			M	11.7	15.8	20.5	22.2	27.9	34.2	41.1	36.6	44.0	52.3	61.0	70.6
			A_s	1.36	1.57	1.78	1.95	2.16	2.39	2.61	2.58	2.82	3.06	3.30	3.54
		1000	t_1	2.25	2.63	3.00	3.00	3.38	3.75	4.13	3.75	4.13	4.50	4.88	5.25
			M	15.1	19.7	25.5	27.7	34.9	42.7	51.5	46.1	55.5	65.5	76.5	88.5
			A_s	1.73	1.99	2.25	2.48	2.77	3.05	3.33	3.31	3.62	3.92	4.22	4.53
		1125	t_1	2.42	2.82	3.22	3.22	3.63	4.03	4.44	4.03	4.44	4.85	5.25	5.65
			M	17.8	23.8	30.8	33.1	42.4	51.8	62.2	56.1	67.4	79.7	93.4	108.0
			A_s	2.13	2.44	2.77	3.05	3.41	3.75	4.11	4.10	4.48	4.86	5.25	5.61
	3000 10	1050	t_1	2.06	2.41	2.76	2.76	3.10	3.44	3.78	3.44	3.78	4.13	4.47	4.81
			M	14.1	19.0	24.6	26.7	33.5	41.0	49.3	43.9	52.9	62.8	73.3	84.7
			A_s	1.63	1.88	2.16	2.34	2.60	2.87	3.13	3.10	3.38	3.68	3.96	4.25
		1200	t_1	2.25	2.63	3.00	3.00	3.38	3.75	4.13	3.75	4.13	4.50	4.88	5.25
			M	18.1	23.7	30.6	33.3	41.9	51.2	61.7	55.3	66.5	78.5	91.7	106.1
			A_s	2.08	2.39	2.70	2.97	3.32	3.66	4.00	3.97	4.35	4.70	5.07	5.44
		1350	t_1	2.42	2.82	3.22	3.22	3.63	4.03	4.44	4.03	4.44	4.85	5.25	5.65
			M	21.3	28.6	37.0	40.5	50.9	62.1	74.6	67.3	80.9	95.6	112.0	129.5
			A_s	2.56	2.92	3.32	3.66	4.09	4.50	4.93	4.91	5.37	5.83	6.30	6.74

Notes:

(1) In the above table, the slab thicknesses t_1 are equal to kd. Where these values are less than 3", it is suggested that the moments and steel areas be used as indicated, but the values for t_1 be made 3" for construction purposes.

(2) When the area of steel involves bars larger than 1"ϕ, it is also suggested that the thickness t_2 be increased for better embedment.

(3) For simplicity, d' has been assumed as 1". When bar sizes or building codes require an increase in this dimension, the difference may be added to the overall depth, D, and the tabulated values used without modification.

Addendum to a paper by I. E. Morris:

Getting More for Our Concrete Dollar*

Since publication of the paper, tests of a model slab have provided some interesting data which demonstrate the inherent strength of the arched slab.

The ends of the model slab (Fig. A) were placed on two 6-in. WF beam sections resting on a heavily reinforced concrete pavement. A third WF section was placed transversely across the top of the slab at its center. A screw jack and a precision 8000 lb proving ring were centered on the transverse beam and brought to bear against an overhead steel frame anchored to the concrete pavement. A micrometer dial gage was placed between the transverse beam and the pavement to measure the deflection of the slab under load.

*ACI JOURNAL, Jan. 1951, *Proc.* V. 47, p. 389. Disc. 47-30 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47.

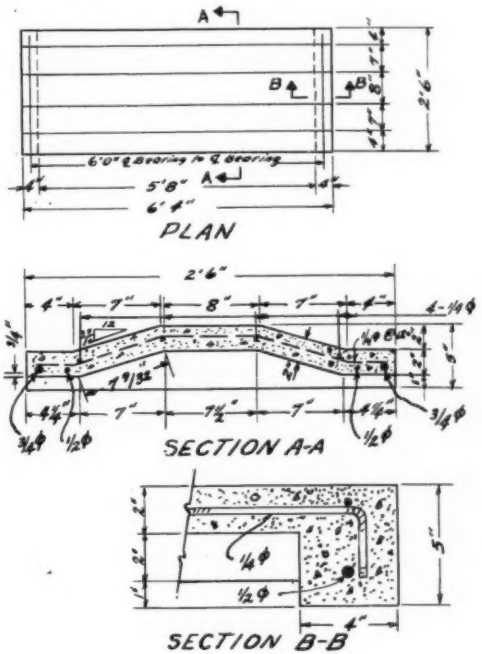


Fig. A—Details of test slab

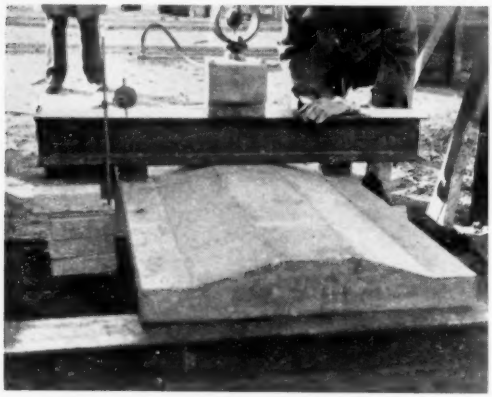


Fig. B—Arrangement of apparatus for testing model slab

A second micrometer dial gage was placed against the side of the slab to measure lateral deflection of the slab (Fig. B).

The load was applied by the screw jack and measured by the proving ring in increments of approximately 400 lb, increased every half minute. Deflection corresponding to each load was read on the micrometer dial gage (Table A).

Using 3000-lb concrete, we figured that a concentrated load of 1300 lb would, by the method of analysis set up in the paper, produce an extreme fiber compressive stress of 1350 psi in the concrete, the maximum permitted by the ACI Code. However, a load of 7500 lb was required to rupture the slab (Fig. C and D), which means that the breaking load was nearly six times the design load. Another interesting point is that no visible cracks appeared until the load reached 6600 lb. Moreover, the lateral deflection was not enough to measure accurately.

TABLE A—DEFLECTION OF BEAM WITH LOAD

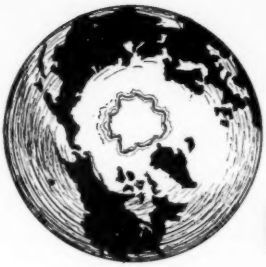
Load, lb	Vertical deflection, in.
0	0
400	0.050
750	0.094
1120	0.146
1540	0.214
1950	0.280
2300	0.340
2700	0.380
3000	0.410
3450	0.450
3850	0.485
4250	0.520
4650	0.555
5050	0.595
5400	0.630
5800	0.670
6200	0.715
6600	0.765
7000	0.820
7150	0.870
7500	1.079

Fig. C—Condition of top of slab at failure under 7500-lb load

Fig. D—Underside of slab at failure

Investigations since publication of the paper indicate that the slab functions in such a manner as to counteract any lateral thrusts which may result from arch action.

In view of these investigations and the action of the test slab, it appears that the original assumptions with respect to lateral thrusts were quite conservative.



Letters from Readers*

BY WAY OF SYNOPSIS

Possible corrosive effect of fly ash in concrete is analyzed by J. L. Gilliland.

Eivind Hognestad gives background information and a bibliography on the development of the plastic theory of design.

The use of cellular shells is discussed by Paul Rogers.

Data from Miles F. Wortham shows the economy of reinforced concrete construction.

Manuel H. Ly tells about a rising appreciation of concrete buildings in Manila.

F. N. Menefee and D. H. Pletta consider the destructive effect of corrosion of reinforcing steel in concrete.

A concrete course for college students is described by W. R. Mason.

Relationship of Fly Ash and Corrosion (LR 47-60)†

In the September 1950 issue of the Journal a reader asks about the possibilities for chemical and electrolytic corrosion of metallic members in concrete where fly ash has been used as an aggregate or a partial substitute for portland cement.

It appears that he is concerned with the possible presence of sulfides in the fly ash, so his question can be answered by investigating the likelihood of sulfur in fly ash being present as sulfides.

The chemical analyses of fly ashes from a number of sources are given by R. E. Davis (ACI Journal, May-June 1937, *Proc.* V. 33, p. 588). The total sulfur and sulfur compounds, computed as SO_3, range from 0.42 to 2.34 percent. These analyses were made by oxidizing all sulfur to the sulfate form. Analyses made at the Bureau of Reclamation laboratories on fly ashes from a limited number of sources indicate that the sulfur in the samples studied did not require oxidation, because the sulfur was already in the sulfate form. This is not totally unexpected, since boilers are operated with an air excess to form an oxidizing atmosphere.

It seems probable, then, that the sulfur is present as calcium sulfate and alkali sulfate, and would have an effect similar to the gypsum in the cement. Actually, the amount of sulfate in fly ash is about the same as the sulfate in cement.

The rate of corrosion of steel is greatly affected by the pH of the environment. At high pH, the rate of corrosion is very small. Therefore, in the highly alkaline condition prevailing in moist concrete, little corrosion will be expected, regardless of whether or not fly ash is present.

J. L. Gilliland
Head, Chemical and Cement Laboratory, U. S. Bureau of Reclamation, Denver, Colo.

*A part of copyrighted Journal of the American Concrete Institute, V. 22, No. 5, Jan. 1951, *Proceedings* V. 47. Separate prints of the entire Letters from Readers Section are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†See also ACI Journal, Sept. 1950, *Proc.* V. 47 p. 74.

Skeptism Regarding the Plastic Theory of Design (LR 47-66)*

The writer has studied H. M. Hadley's letter to the editor, ACI JOURNAL, Nov. 1950, pp. 257-260, with great interest. This is certainly not the first skepticism that has been recorded in the literature regarding the plastic theories of design. As far as the writer knows, however, no previous criticism has ever claimed that plastic theories do not reflect the ultimate capacities of reinforced concrete members in a more realistic manner than does the straight line theory. The discussion in the past has been concerned mainly with the difference in philosophy between the ultimate load design and the familiar concept of working stresses, not with the *validity* of plastic theories. Hence, Hadley's discussion is indeed unique.

It is felt that Hadley's conclusion that "the 'plastic theory of design' for concrete flexural members is or should be as flat as a punctured balloon" is entirely unjustified. This conclusion is further based on such arguments that the writer considers it difficult to meet them in detail. It is stated, for instance, that "The column formulas which are used resulted from the long-time column tests at Lehigh University and the University of Illinois and are in accordance with those tests." This statement is not in accordance with the tests reported and the conclusions reached in Bulletin 267 of the University of Illinois which contains some of the tests referred to. This type of argument can therefore not be met with normal logic.

It is pointed out, on the other hand, that the research work backing up C. S. Whitney's plastic theory "appears to have consisted wholly of the work of two seniors at the University of Wisconsin," and that Jensen's "pattern of stress distribution is essentially the same as Whitney's." These arguments may be interpreted as meaning that the plastic theories of design were conceived by Whitney and Jensen on the basis of merely a few month's research done by two undergraduate students. The writer considers it proper to show that this is certainly not so.

Coignet and Tedesco[1] who first presented what is now generally referred to as the straight line theory realized that this theory does not strictly reflect the properties of reinforced concrete. They claimed, however, that their theory had great advantages in its mathematical simplicity. It tied the new material in to the well-known Navier's theory of bending, and it was accurate enough for design purposes at their stage of knowledge (1894) when safety factors for concrete in compression were about 5. Many plastic theories of design also appeared in these early years, most important of which may be the work by R. M. v. Thullie,[2,3] W. Ritter,[4] and A. N. Talbot.[5] Most of these early students of plastic theories conceded, however, that the straight line theory at that time was most suitable for design purposes. Hence, this latter theory became established at the turn of the century, and a very rapid development of the use of reinforced concrete as a construction material followed. It should be noted, on the other hand, that the limitations of the straight line theory already were discussed intelligently at this time. Furthermore, it is significant that the first plastic theories grew out of the same basic studies[6,7] as did the straight line theory.

The plastic theories were gradually improved as more knowledge about the basic properties of reinforced concrete was unveiled, and this knowledge led to periodic changes in the standardized straight line theory. The rectangular stressblock as perfected by C. S. Whitney is certainly not a sudden brainstorm from New York. This stressblock was developed and used by a number of other engineers.[8 *to* 14] Similarly, V. P. Jensen's theory is no isolated case referred to the geographical location Urbana, Illinois. A multitude of investigators have based their studies on the same type of basic assumptions.[15 *to* 32] These *few* selected references cover most of the geographical area generally referred to as the western civilization, and they indicate that the plastic behavior of reinforced concrete *is* "A natural phenomenon observed, say in Berlin, Germany, or in Chicago, Illinois, (which) can under like conditions similarly be observed in Seattle, Washington, or in Denver, Colorado or anywhere else in the world." It may also be pointed out that

*See also ACI JOURNAL, Nov. 1950, *Proc.* V. 47, p. 257.

several of these references contain experimental verification of such plastic theories as carried out, not by two undergraduate students, but by some of the most outstanding laboratories in the world and further by those same great engineers who developed and extended the straight line theory.

The question today is not whether or not the plastic theories are technically sound, but rather how far such theories must be developed in detail and whether or not the construction industry is ready to use an ultimate design procedure to an advantage in dollars and cents.

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Cellular Shells (LR 47-69)

The cellular shells described by F. E. Wolosewick* are commonly used in power plant construction. Many power stations are built near the riverbanks to gain easy access to condenser cooling water. The soil at such locations seldom has the bearing capacity to support the heavy column loads. Quite often the rock, or good load bearing strata, can only be found 60 or 80 ft below the surface. In such cases, hollow cellular caissons can be utilized with economic advantage.

The moment and shear analysis of such caissons is extremely difficult. This is particularly true for elongated cross sections with one or more partitions or stiffeners. Ordinary office practice seldom permits the time necessary for the solution of numerous simultaneous equations. Consequently, short cuts and approximate analyses are often used, resulting in uneconomical construction.

Wolosewick should be congratulated for the successful solution of this difficult problem and for his concise charts, which will indeed be a helpful tool for the practical designer.

PAUL ROGERS, Structural Engineer, Chicago, Ill.

Economy in Reinforced Concrete Construction (LR 47-70)

In designing heat exchanger structures and tower access platforms for an oil refinery a study was made of reinforced concrete and fireproofed steel structures using prevailing prices at the Pan American Refinery in Texas City, Texas. There are several points of interest in this comparison that influenced the unit prices.

1. No shoring is required for the steel fireproofing and the shoring and forms for the concrete structure were re-used on the second level.

2. Cantilever construction was used in the reinforced concrete structure for access platforms to the heat exchangers thus eliminating one row of columns used in the structural steel structure.

*See "Flexure of Cellular Cells," ACI JOURNAL, Dec. 1949, *Proc.* V. 46, p. 249.

TABLE 1—REINFORCED CONCRETE STRUCTURE

Item	Unit cost	Quantity	Total
Structural steel, lb			
Fabricate	$0.09	17,560	$1580.40
Erect	0.025	17,560	439.00
Paint	0.015	17,560	263.40
Reinforced concrete			
Concrete, cu yd	25.00	22	550.00
Forms, sq ft	0.70	1410	987.00
Reinf. steel, lb	0.14	5600	784.00
Patching, sq ft	0.07	1410	98.70
Total cost			$4702.50

TABLE 2—FIREPROOFED STRUCTURAL STEEL STRUCTURE

Item	Unit cost	Quantity	Total
Structural steel, lb			
Fabricate	$0.09	40,830	$3674.70
Erect	0.025	40,830	1020.75
Paint	0.015	40,830	612.45
Fireproofing			
Concrete, cu yd	22.50	30	675.00
Forms, sq ft	0.45	2460	1107.90
Total cost			$7090.80

3. The foundation for both types of structures would be approximately the same due to the same equipment loads and fireproofed structure weighing approximately the same as the reinforced concrete one.

4. The specifications required that all structures supporting equipment be fireproofed, which in our design was elevation 119 ft.

5. Above this elevation a steel frame with monorails was used to facilitate the installation of the heat exchangers and the removal of the tubes.

6. From Tables 1 and 2 it may be seen that a saving of 33.68 percent was accomplished by the use of a reinforced concrete structure.

MILES F. WORTHAM,
Structural Engineer, Tellepsen Construction Co.,
Houston, Texas

Reinforced Concrete in Manila—1950 (LR 47-71)

Before the war there were few new reinforced concrete buildings in Manila except those put up by the government of the Philippines or by businessmen along busy main streets. After the war, land owners began to awaken to the many good qualities of concrete as a building material. While most of the city was leveled during the liberation of Manila, the few reinforced concrete buildings stood as guiding landmarks in a ghost city. During the rehabilitation of the city, owners of reinforced concrete buildings were able to repair and utilize them by simply throwing a roof over the hollow structure. With the permanence of concrete brought home so forcefully, prospective builders suddenly realized the ability of concrete buildings to withstand time and change, with the effect that they are starting to build houses of this material. So Manila is now dotted with tall reinforced concrete buildings, both commercial and residential, making it a truly modern city.

Because of the infrequent concrete construction before the war, there is today a stagnation in concrete design. Another contributing factor to this stagnation was the lack of public interest and the lack of realization of the adaptability of concrete for buildings. This has resulted in a scarcity of concrete technicians and yet these few technicians are not properly paid. The public seems to be still suffering from a wrong notion that a concrete technician, since he is new in the field, should render his services free.

Three phases connected with reinforced concrete, design, inspection during construction and estimating, illustrate the lack of concrete knowledge.

We find for example that present design must conform with obsolete building codes as embodied in the city ordinances, *viz.*, direct stress in concrete—extreme fiber stress in flexure—0.3 f_c' or 600 psi and allowable tensile stress for reinforcement (imported U. S. steel), 16,000 psi. Because of this limitation, whenever I want to design a structure in accordance with the stresses given above, I have to use original design formulas, and not design tables issued by ACI or as published in engineering books and pamphlets. This in spite of the fact that locally manufactured cement conforms with ASTM Standards and the reinforcing steel

is imported from the United States. Present ordinances require that concrete buildings be designed strictly in accordance with the 1921 (it may mean 1916) "Report of the Joint Committee on Standard Specification for Concrete and Reinforced Concrete." This provision has been in the book, if I am not mistaken, for the last 15 years.

As for concrete inspection during construction we can venture the view that there is no inspection at all. Although the ordinances provide for an inspector who must be an engineer or architect, the house owner, to save money, gets around it by using the name of the designer as inspector, and forgets to talk with the designer about the necessary compensation. The obvious result is that the concrete building is finished without inspection.

Structural engineers as a rule do not make good here. Architects generally charge 1 percent of the cost of the building for plans, and of this he pays 20 percent for structural plans. Comparing this rate with other places, we find it quite low. As a consequence of the lack of demand before the war, many engineers drifted into other professions, so that most engineers and contractors were not prepared for the concrete boom after the war. Since the engineers had a lot of book knowledge but no actual experience, the effect was that bids for concrete structures in 1946 and 1947 varied from 30 to 40 percent. In fact, there was a standing joke among contractors during the early part of the liberation that it was only when you made a mistake in your computation of bids that you got the work. At present a group with more experience and "know-how" has moved in and we are getting better and closer bids.

MANUEL H. LY, Consulting Engineer, Manila, Philippines

Destructive Effects of Corrosion (LR 47-72)

The paper "Corrosion Protection of Thin Precast Concrete Sections," by Pletta, Massie and Robins* is an important contribution on this subject. The basic question of corrosion, its disastrous effects on concrete and how to prevent it, is important. The tendency is toward thinner sections, which usually means less cover.

We have all seen concrete girders, columns and beams where corrosion has split the concrete and exposed the steel to worse corrosion and eventual failure of the structure in many cases. Some structures suffering from this disease of concrete can be saved by proper application of shotcrete. The writer recently saw a reinforced concrete bridge between the mainland and Clearwater Beach, Fla. Corrosion has exposed a large percent of the moment steel in this causeway. It may have progressed to a point where complete replacement will be necessary.

In Fig. 1 and 2, an abandoned metal stairway on the outside of one of the buildings at the University of Michigan dramatically shows the tremendous force set up by the expansion of corroding steel, or possibly that set up by freezing of water within the finely divided particles of iron oxide.

Fig. 1 is taken from the outside of the stairway. It shows the flange *A* on the stair rail and the flange *B* on the steel I-beam. It also shows the metal braces *D* to the hand rail above. Fig. 2 is taken from underneath the stairway. It shows the flange *A* on the stair rail and flange *B* on the I-beam support.

Fig. 1—Effects of corrosion on exposed metal stairway

*See ACI JOURNAL, Mar. 1950, *Proc.* V. 46, p. 513.

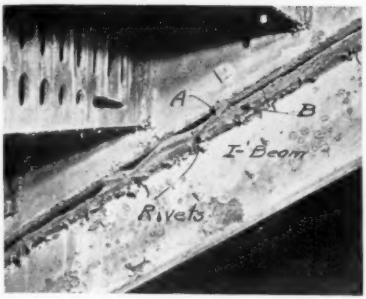

Fig. 2—Effects of corrosion on underside of exposed metal stairway

The flanges from the stair rail and supporting I-beam were riveted together at intervals. The rivets have held, but corrosion between the flanges has forced them apart between rivets and in some places the flange from the stair rail above is torn away, as shown by both figures. The force was so great that the braces *D* in Fig. 1 were broken in tension.

These figures show in a qualitative way something of the force set up by corroding steel in concrete. They plainly indicate forces that concrete will not withstand.

Better concrete and prestressing will to some extent offset the reduction in cover arising from thinner sections, but researches such as the one described by the authors will be valuable to the profession in that they will be helpful when it comes to setting limits on cover for reinforcing under the many conditions that arise in concrete construction.

F. N. MENEFEE, Professor of Engineering Materials, University of Michigan, Ann Arbor, Mich.

The corrosion of ferrous metals, as Menefee points out is accompanied by a considerable change in volume, and this expansion is capable of exerting considerable force when confined in reinforced concrete. If and when such corrosion can begin and continue, disintegration of the concrete should be expected.

The investigation described in the authors' paper sought to determine the protective effect of uncracked cover over the reinforcing. The results indicate that coverage as thin as $\frac{3}{8}$ in. will prevent corrosion for at least one year, provided the mix is plastic and dense. Although corrosion was expected in all of the panels tested, it just did not materialize. The effects of cracking and time may alter this minimum required thickness, but there is no quantitative data available yet on which to base a statement.

D. H. PLETTA, Professor and Head, Applied Mechanics Dept., Virginia Polytechnic Institute, Blacksburg, Va.

Concrete Course for College Students (LR 47-73)

At the University of Washington we attempt to give the student an intimate contact with plain concrete rather than a course in the routine standard tests. Two two-hour periods per week are used for laboratory work and a third two-hour period for lectures and field trips. Field trips are taken to a ready-mix plant and to a construction project. The PCA film, "Drama of Portland Cement," is used to acquaint the student with the manufacture of portland cement. The Bureau of Reclamation *Concrete Manual* is used as a text.

The course is designed to first acquaint the student with the properties of concrete, leading up to an application of his knowledge in designing a mix near the end of the quarter.

Laboratory work includes the following experiments:

1. An age experiment in which strength-age tests are made on standard, high-early and Type II cements. The cylinders are tested at 1, 2, 7, 14 and 28 days. Beams are also made and tested at 28 days to determine the modulus of rupture.
2. A slump experiment in which all factors are kept constant except the amount of paste which is used to vary the slump. Cylinders made of concrete with slumps of 0, 3, 5, 6, 7 and 9 in. are tested at seven days. Students find the effect of slump on strength, workability and cement content.
3. A water-cement ratio experiment in which all factors are kept constant except

the water-cement ratio. The amount of the various cement and water pastes are varied to obtain a 2-in. slump on all specimens. Tests are made at seven days on cylinders made of concrete with W/C of 0.35, 0.45, 0.55, 0.65, 0.75 and 0.85 to find the effect of W/C on strength, workability and cement content.

4. An experiment in which all factors are kept constant except the percentage of sand. The amount of paste is varied to obtain a 2-in. slump on all specimens. Seven-day cylinders with 10, 30, 35, 45, 60 and 80 percent sand are tested to find the effect of the percentage of sand on strength, workability and cement content.

5. An experiment in which all factors are kept constant except the gradation of the sand. The paste is varied to obtain a constant 2-in. slump. Gradations of sand of FM 4.6, 4.1, 3.44, 2.89, 2.39 and 1.93 are used in concrete cylinders tested at seven days. Students find the effect of the gradation on strength, workability and cement content.

6. A curing experiment in which curing conditions are varied from freezing after one hour to steam curing for eight hours. Cylinders for each of the six curing conditions are tested at seven and 14 days to compare strength results with those of the first experiment.

7. The more important ASTM aggregate tests are made. Included are sieve analysis, unit weight, specific gravity and absorption.

8. A concrete mix is designed to tie together the preceding experiments. Students are given a design strength ±100 psi and a design slump ±½ in. and the requirement that the mix be as economical as possible. The time of test is reduced to five days so an interpolation of preceding experimental data is necessary. It also enables the student to test specimens a few days before his next trial mix.

It is believed that this course gives the student the feel of plain concrete as he has mixed and tested many different mixes.

W. R. Mason, Assistant Professor of Civil Engineering, University of Washington, Seattle, Wash.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

The Magnel-Blaton system of prestressing

D. H. May, *The Reinforced Concrete Review* (London), V. II, No. 2, Apr. 1950, pp. 135-139

Reviewed by C. P. Siess

A brief description of the Magnel-Blaton system of prestressing. The possible effects of loss of wire section due to corrosion are discussed, and mention is made of a fire test on a prestressed post-tensioned beam.

A continuous bridge in prestressed concrete

The Reinforced Concrete Review (London), V. II, No. 2, Apr. 1950, pp. 139-144

Reviewed by C. P. Siess

Description of the design and construction features of a two-span continuous prestressed concrete bridge at Sclayn, Belgium. The Magnel-Blaton system was used. Includes three photographs of the bridge under construction and two plates of design details.

Rapid computation of arch bridges (Calcolo rapido dei ponti ad arco)

Domenico Priolo, *Giornale del Genio Civile* (Rome), V. 87, No. 12, Dec. 1949, pp. 645-656 and V. 88, No. 2, Feb. 1950, pp. 103-113

Reviewed by Gennaro Mianulli

This is a rapid method and a new tool for the solution of fixed arches. It is based on the exact squaring of integrals appearing in equations, expressing redundant conditions, referred to the elastic center. The exact integration which eliminates the necessity of numerous summation tables is made possible by reasonable assumptions on the shape of the arch curve and variation in moment of inertia of the cross section.

The heat and sound insulation of reinforced concrete buildings

Charles W. Glover, *The Reinforced Concrete Review* (London), V. II, No. 2, Apr. 1950, pp. 93-117

Reviewed by C. P. Siess

A relatively condensed discussion of the principles and practice of thermal and sound insulation of concrete buildings. Includes tables of thermal properties, and contains many illustrations of types of construction used to provide insulation. References to other papers on this subject are given.

Study of curved walls for tanks and a simple method of design (Metodo semplice per lo studio delle lastre curve)

Odone Belluzzo, *Giornale del Genio Civile* (Rome), V. 87, No. 1-2, Jan.-Feb. 1949

Reviewed by Gennaro Mianulli

This is the last of a series of articles on the analysis of reinforced concrete thin wall tanks with multiple curves having a common edge.

The design uses elastic coefficients developed in Part 3 and membrane deformations considered in Part 2.

Architectural design in reinforced concrete and its pioneers (Formgivningen og dens pionerer indenfor jernbetonbyggeriet)

Jesper Tvede, *Beton-Teknik* (Copenhagen), V. 16, No. 2, Sept. 1950, pp. 41-57

Author's Summary

The author points out that the appearance of new materials raises problems in connection with the natural forms for these materials, in contrast to the more obvious application of the already prevailing forms.

*A part of copyrighted Journal of the American Concrete Institute, V. 22, No. 5, Jan. 1951, *Proceedings* V. 47. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

The development of architecture in relation to communal development over the past century is touched upon, and mention is made of the foremost pioneers of reinforced concrete architecture and their work.

The rapid design of reinforced concrete slabs and beams

W. SEFTON, *Civil Engineering and Public Works Review* (London), V. 45, No. 530, Aug. 1950, pp. 506-508
Reviewed by CHARLES W. DOHN

Two charts are presented, one for the design of reinforced concrete slabs and the other for the design of reinforced concrete beams including shear reinforcement. This type of design chart produces a vivid picture of design ranges and is sufficiently similar to design values commonly used so as to be universally adaptable.

Observations derived from testing an actual arch model built of expansive cement concrete (Osservazioni sulla documentazione sperimentale di un arco in calcestruzzo di cemento espansivo)

GIUSEPPE RINALDI, *Giornale del Genio Civile* (Rome), V. 88, No. 1, Jan. 1950, pp. 12-22
Reviewed by GENNARO MIANULLI

The author describes a testing procedure for concrete arches using expansive cements, stating the purpose and conclusions obtained from an actual model. The calculations and diagrams were taken from "L'utilisation des cements expansif," by H. Lossier, the inventor of this cement. The tests were made to verify Lossier's findings.

Experimental investigation on failure in reinforced concrete units under bending (in French)

R. CHAMBAUD, *Final Report*, International Association of Bridge and Structural Engineering, 1948, pp. 671-686
APPLIED MECHANICS REVIEWS
Oct. 1950

Authors carried out experimental study of failure in reinforced concrete parts under bending. Tests included 82 beams of rectangular section and were carried out with great care. They reach the decisive conclusion that classical theory considerably underestimates resistance of compressed concrete in realm of high percentages of stressed steel, and that use of compressed steel is practically never justified.

They propound an elastroplastic theory for calculating concrete units under bending, with a constant safety factor. That theory, which proved to agree with the experiments carried out by the authors, leads to a very important economy in strongly reinforced parts.

Caissons as dam foundations

F. F. FERGUSSON, *Civil Engineering and Public Works Review* (London), V. 45, No. 529, July 1950, pp. 452-453
Reviewed by CHARLES W. DOHN

This article describes concrete caissons to be used for both earth fill and concrete gravity type dam foundations where depth of alluvium overlying bed-rock is considerable. Such construction is projected by the Damodar Valley Corp. for dam sites on Indian rivers. The plan of a caisson, a section through an earthen embankment dam with caisson cut-off and a section through a concrete gravity type dam surmounted on two rows of concrete caissons are included.

The conjugate frame as a tool for evaluating deflections

GLENN MURPHY, *Proceedings*, Seventh International Congress of Applied Mechanics, V. 1, 1948, pp. 131-138
APPLIED MECHANICS REVIEWS
Oct. 1950 (Phillips)

The author gives an extension of conjugate-beam procedure to frames. He uses a geometrical analysis to show analogy between deflections of original frame and bending moments in conjugate frame. If original frame is fixed at both ends, conjugate frame is the analogous column of Cross' "column analogy." An illustrative problem is given. It is pointed out that conjugate frame may be used as a tool in solution of statically indeterminate structures.

The corrugated concrete arch—Part 2

J. H. DE W. WALLER, *Civil Engineering and Public Works Review* (London), V. 45, No. 529, July 1950, pp. 449-450
Reviewed by CHARLES W. DOHN

A number of illustrated examples of structures, including a domed form, built according to the corrugated concrete arch principle are described. New interest is aroused in that ancient and most efficient form of structure—the true arch. The corrugated concrete arch is a form of structure whose efficiency has not yet been exploited on a large scale in this country. Extremely thin corrugated simple arch shells of long span remain in compression even under the most severe conditions of live load.

Use of accelerators in the manufacture of insulating slabs of cement-fiber composition (L'emploi des accelerateurs dans la fabrication des plaques isolantes en fibres agglomerees au ciment)

M. THUILLEAUX, *Revue des Materiaux* (Paris), No. 419, Aug. 1950, pp. 270-1

Reviewed by PHILLIP L. MELVILLE

Insulating slabs are made of wood fibers (or equivalent) bonded with a cement slurry or emulsion. $CaCl_2$ is needed as an accelerator. About 8 percent of the salt by weight of cement is used depending on materials, temperatures and manufacturing process. Partly covered reinforcement will corrode. The slabs have good fire protection properties.

Chemical deterioration of concrete (Kjemisk forvitring av betong)

JOHS. RUTLE, *Teknisk Ukeblad* (Oslo), V. 97, No. 20, May 18, 1950, pp. 383-392

Reviewed by EIVIND HOGNESTAD

A brief discussion of the mineral constituents and hydration process of portland cement is presented as an introduction. The chemical stability of concrete in contact with various chemicals is discussed under the sub-headings: bases, salts, sea water, alum shale, acids, carbonic acid, and organic acids. Precautions and remedies against chemical attacks are presented emphasizing construction methods, special cements, pozzolana, air entrainment, coatings and autoclave curing.

A sulfate susceptibility test for portland cements

W. C. TAYLOR and R. H. BOGUE, *Journal of Research*, National Bureau of Standards, V. 45, No. 3, Sept. 1950, p. 223

Reviewed by C. C. FISHBURN

A procedure has been developed by which portland cements may be given ratings indicative of their resistivity to chemical attack by sulfate-containing solutions. The test, which may be performed in about a day, reveals the intrinsic resistivity to such attack by the cement itself as contrasted with the usual performance tests on mortar or concrete specimens, which require periods of weeks or months. The method involves the measurement of the amount of dissolved SO_3 rendered insoluble when cement is shaken with limewater for a stated period. The test has been examined critically and extensive correlations made with performance tests on laboratory and commercial portland cements.

New manual for construction of silos (Ny vejledning i optoerelse af siloer)

E. V. MEYER, *Beton-Teknik* (Copenhagen), V. 16, No. 2, Sept. 1950, pp. 58-62

AUTHOR'S SUMMARY

The Royal Danish Agricultural Society has published new instructions for silo construction. The instructions cover 3 types of silos, *viz*: tower silos, semi-tower silos and pit silos, which may be either monolithic concrete constructions or built of elements of concrete or tile. Both materials and building methods are described. Calculations are based on a water-pressure equivalent to three-fourths of total height for beet-tops, grass and clover silage, but to full height in the case of beet-waste silage.

Rectangular slabs elastically supported along its edges under any condition of loading (La lastra ettangolare sostenuta elasticamente lungo tutto il contorno sottoposto a condizione qualungue di carico)

PIETRO POZZATI, *Giornale del Genio Civile* (Rome), V. 87, No. 1-2, Jan.-Feb. 1949, pp. 55-67; V. 87, No. 5, May 1949, pp. 232-243; V. 87, No. 7-8, July-Aug. 1949, pp. 361-371; V. 88, No. 3, Mar. 1950, pp. 145-155

Reviewed by GENNARO MIANULLI

The author shows a procedure of analysis for flat plates elastically supported along its edges. Various conditions of supports and loadings are investigated and illustrated by examples. Marcus' method of finite differences is followed; its limitations are overcome by the use of solutions in simple series.

Design of prestressed tanks

J. M. CROM, *Proceedings* ASCE, V. 76, Separate No. 37 Oct., 1950, 20 pp. $0.50

AUTHOR'S SUMMARY

In the past, the safe design of circular structures has been extremely difficult, mostly because exact knowledge concerning shrinkage and plastic flow of concrete has been lacking. This paper contains new data for evaluating these phenomena and describes methods that make practicable the use of the high strength of cold-drawn steel wire. Such developments have made it possible to design tanks and other large circular structures on a rational basis, with the assurance that an adequate prestress will be maintained to eliminate cracking of concrete and that important reductions will be made in the weight and in the quantities of critical raw materials required for their construction.

Recent research on prestressed concrete (in German)

ERICH FRIEDRICH, *Final Report*, International Association of Bridge and Structural Engineering, 1948 pp. 359-366

APPLIED MECHANICS REVIEWS Oct. 1950

With prestressed systems two varying load-stages have to be taken into consideration when laying down a permissible load. First stage is the projection load. This depends mainly on the extent of prestressing. Second stage is the rupture load, which depends upon strength of building material. Report mentions tests which show the connection between the two load stages and measuring processes.

Some technical and economic considerations of the Swedish 1949 specifications (Nagra tekniska och ekonomiska synspunkter pa 1949 ars betongbestaemmelser)

VALTER HASSELBLAD, *Cement och Betong* (Malmoe), V. 25, No. 1, Mar. 1950, pp. 61-70

Reviewed by EIVIND HOGNESTAD

These new specifications allow six different qualities of concrete, five qualities of reinforcement and three classes of construction. In principle, the allowable stresses increase with increasing quality of materials. To use such high quality materials, a high class of construction work is demanded. Furthermore, the use of high steel stresses are permitted for high concrete qualities only.

The design and construction cost of a pier and a building is studied. On the basis of economy, the new specifications lead to hard grade reinforcement (yield point over 50,000 psi), a high concrete quality, and a medium class of construction with medium strictness of inspection.

Determination of modulus of elasticity of rocks on site (Determination du module d'elasticite des roches en place)

P. HABIB, *Annales de L'Institut Technique du Batiment et des Travaux Publics*, New Series, No. 145, Sols et Fondations, No. 3, 1950

Reviewed by IVAN M. VIEST

The author proposes a method for determination of the modulus of elasticity of rocks by testing on the site. Laboratory tests on concrete blocks, made to prove the reliability of this method, are described. The blocks were loaded through discs and deformations of the concrete surface were measured. Modulus of elasticity is determined from these data and the Boussinesq equations for deflections of semi-infinite solids. Thus found modulus of elasticity of concrete is compared with values determined from the tests of standard cubes and with sonic modulus. The remaining parts of the paper deal with the modulus of elasticity of rocks.

Precast reinforced concrete rigid frames (Charpentes en beton arme)

W. MUNZ, *La Technique Moderne-Construction* (Paris), V. 5, No. 8, Aug. 1950, pp. 251-253

Reviewed by ALEXANDER TURITZIN

The author describes three industrial plants built of precast reinforced concrete sections of the rigid frame type, as follows: single-span gable-roof frame, double-span gable-roof frames and four-span gable-roof frames with two frames of low roof and two frames of high roof construction. The columns of the four-span gable-roof frames were designed to carry crane loads.

Rigid frame members were designed so that they would be joined together at points of minimum stresses. They were prefabricated of vibrated concrete, carefully proportioned and assembled in the field to form structures of high rigidity.

Frames of a maximum span of 20 m and a height of 10.7 m were used with heaviest individual members weighing 3600 kg. All frame columns were rigidly fixed at their bases by heavy footings. In one instance caisson foundations were used to provide a fixed base in a weak soil strata.

Comparative study of an overdeterminate frame through different methods used in models (Etude comparative d'un portique hyperstatique par differentes methodes utilisees en maquettes)

JEAN ASSINAIRE, *Cahiers du Centre Scientifique et Technique du Batiment* (Paris), No. 8, 2 Trimestre, 1950

Reviewed by PHILLIP L. MELVILLE

On an overdeterminate structure two experimental methods may be used. The solution may be obtained from a direct method by applying actual forces and measuring stress, strain and reactions or by indirect method by applying different forces. In the first case a photoelastic study on a polarized flexiglass model or Rieckhof's device (Nupubest) were used. Either method enables the determination of points of zero moment which is of limited value as they only apply to one given case. In the second case lines of

influence were determined by using Begg's deformeter or the Nupubest. Results were very encouraging especially in the case of Beggs deformeter. Theoretical study is appended as a matter of reference.

Practical design of solid-barrel, reinforced-concrete skew structures

BERNARD L. WEINER, *Proceedings*, ASCE, V. 76, Separate No. 39, 1950, 22 pp. $0.50

AUTHOR'S SUMMARY

No new theories are presented. The purpose is rather to show the possible simplifications in the application of existing theory. These simplifications have been tested in practice. A solid-barrel skew frame can be designed with little more work than that required for the design of a similar right-angle frame. Theoretically, it is true that a skew frame must be analyzed in three dimensions, whereas a right-angle structure can be analyzed in a single plane. It is to be expected, therefore, that there will be stresses in the former type of frame which either do not exist in the latter type or exist only to a very moderate extent. Although these additional stresses (often referred to as "skew stresses") are important, provision can be made for them, for practical design purposes, with very little additional work.

Wind-load standards in Europe

JOHN W. T. VAN ERP, *Proceedings*, ASCE, V. 76, Separate No. 42, Nov. 1950. 12pp. $0.50.

AUTHOR'S SUMMARY

As a result of a continued drive toward greater economy in structural design, wind-load standards have been more and more refined so that they agree almost exactly with actually occurring conditions. The task of the engineer is to proportion his design to fit the stipulated purpose of the structure with the utmost degree of economy while retaining the required factor of safety. The factor of safety covers a certain number of unknown quantities and it has been the constant endeavor of engineers to acquire a better understanding of these unknowns. Thus, on the one hand, analysis of the structure will have to yield stresses agreeing as closely as possible with those actually occurring whereas, on the other hand, the loads should be the expected ones with the least degree of exaggeration. It is imperative, therefore, that the designer have the best possible knowledge of loading conditions—the least information being available about wind loads. The fundamental dynamic character of wind loads must be translated into static equivalents to give simple data for the use of the designing engineer.

Rationalization of concrete building construction

S. DAHLBERG, *Beton og Jernbeton* (Copenhagen), V. 1, No. 3, Dec. 1949, pp. 71-89

AUTHOR'S SUMMARY

With respect to the rationalization of concrete building construction in Sweden the author says that although no single process or material was definitely changed, the shortage of labor and material during the war inspired Swedish designers and contractors to such a lot of small improvements that, altogether, building costs in Sweden have not increased during the last ten years, in spite of considerable increases in materials and labor costs.

The single fact, which according to Dahlberg is of most importance in this connection, is the steadily increasing number of specially educated concrete job-superintendents and inspectors. Their presence on the job enables the contractor safely and constantly to produce concrete with a 1400 psi working stress.

Dahlberg stresses that thorough rationalization is based partly on effective collaboration among everybody on the job and partly on a very differentiated and quick-working cost control. The three re-estimation diagrams shown seem to indicate that Dahlberg himself has been very sucessful in this respect.

Prestressed concrete (in French)

F. CAMPUS, *Revue de Metallurgie*, V. 5, Dec. 1949, pp. 421-436

APPLIED MECHANICS REVIEWS
Oct. 1950 (Quintal)

Author gives short history of prestressed concrete followed by brief description of basic principles. First part of work will be found useful only by those unfamiliar with the method, but description of tests and of practical realizations which follows is quite interesting. The Hoyer system, in which reinforcement is composed of fully bonded piano wires (Stahlsaintenbeton), has been found quite inferior to method where prestress is applied by bearing plates at each

end of beam. In concluding, the author states that more research is necessary, especially in two different fields: a study of high resistance and high density concrete mixes is of primary importance, as is a better understanding of mechanism of creep or plastic flow in both concrete and cold-rolled reinforcing steel.

A study of reinforced concrete walls and deep beams (Nagra undersoekningar roerande skivor och hoega balkar av armerad betong)

H. NYLANDER and H. HOLST, *Transactions of the Royal Technical University* (Stockholm), No. 2, 1946, 66 pp., 2.00 Sw. Kr.

Reviewed by EIVIND HOGNESTAD

A study of deep beams and walls, which was carried out by means of theoretical analysis, photoelastic studies, and tests of reinforced concrete specimens is reported in detail.

Three major types of failures were found: 1. Local failure of concrete at a support or below a concentrated load. 2. Splitting failure, and failure in diagonal tension. 3. Tension failure of reinforcement in bending. Relatively simple dimensioning rules are developed for each of these types of failure. It is pointed out that cracking of the concrete prior to failure causes such favorable redistributions that dimensioning after theory of elasticity generally gives a very conservative design.

Concrete statue (Betonnen Beeld)

F. CARASSO, *Cement* (Amsterdam) No. 21-22, 1950, pp. 470-471

Reviewed by J. W. T. VAN ERP

At the Olympic Stadium in Amsterdam a 20-ft statue is made entirely of reinforced concrete to blend with the stadium structure in which that same material was largely used. Due to the relatively large size of the statue and the limited time available it was decided to make the statue entirely on location. As concrete is much too hard for chiseling, the following procedure was chosen and proved to be successful.

After his preliminary but accurately detailed small scale model, the sculptor fashioned the full size statue in its final position on site out of clay around an interior wooden skeleton. This clay model, weighing two tons, served as the original for the concrete, and had to be finished down to smallest details.

A reinforced plaster of paris negative mold was then made so that it could be taken apart in sections to permit removal of the clay original.

A reinforcing steel skeleton involving no complicated design mathematics was then erected and the concrete was carefully placed and vibrated in the gypsum mold.

Flame-photometer determination of sodium and potassium in soils and other siliceous materials

W. J. HALSTEAD and BERNARD CHAIKEN, *Public Roads*, V. 26, No. 5, Dec. 1950, pp. 99-104

AUTHOR'S SUMMARY

This article presents a method for using a flame photometer to determine the alkalies —sodium and potassium oxides—in soil and other siliceous material which cannot easily be decomposed by mineral acids. The method is much more rapid than existing gravimetric procedures, and yields results which are equally accurate.

The sample is decomposed by fusion and the melt is leached with hot water to bring the alkalies into solution as chlorides. The concentration of alkalies in the resulting solution is then determined by a flame photometer, using the direct-intensity method.

A wide variety of materials can be analyzed by the method, including soils, sands, rocks, various minerals, ceramic clays, fly ash and pozzolanic, slag, and natural cements. The method is also applicable to the determination of the alkalies in limestones and other calcereous materials.

The angle-web construction method (Die Winkelsteg-Bauweise)

F. OESTREICHER, *Betonstein-Zeitung*, (Wiesbaden), V. 1950, No. 2, p. 35

Reviewed by RUDOLPH FISCHL

A new original construction element for floors and roofs is briefly described. It is formed by an angle-shaped reinforced concrete member in which the reinforcement prevents cracking under dead load conditions. It can be transported and moved as easily as a wooden or steel beam.

Each pair of the concrete angles are placed with webs together and the joints are filled with concrete mortar, thus forming a sequence of T-beams for the floor construction. For roofs individual T-beam combinations are used as rafters.

Large areas of floors have already been built with these units. This construction

method is patented and was honored with the second prize at a public competition where 52 designs for new and more efficient floor construction methods were submitted.

No data concerning size, reinforcement, span and load capacity are given in this article.

Gas concrete and erection method by the Hebel system (Gasbeton und Montagebauweise nach System Hebel)

Betonstein-Zeitung (Wiesbaden), V. 1950, No. 1, pp. 12-15

Reviewed by RUDOLPH FISCHL

This article describes the production of a lightweight concrete slab with aluminum powder as the gas-producing medium. Hebel's special process to pour, cut and prepare the slabs before drying and hardening them under 8 atmospheres of pressure is described in detail. The slab has a weight of 0.8 kg per cu m and an ultimate compression strength of 25 kg per sq cm. The thermal coefficient is approximately the same as for a brick wall 38 cm thick plastered on both sides.

The unit slab for building bearing walls is 2.50 m high, 50 cm wide and 15 cm thick. It is not reinforced. Floors and roof are formed by thinner slabs in connection with precast beams. Buildings up to 3 floors are permitted in Germany. The slabs are tied in each floor by cast-in-place concrete stringers.

Many apartment houses, hospitals and industrial buildings were built in Bavaria after this method and have proved to be very efficient. It is planned to enlarge this building program in the near future.

Manufacturing process of Ferrari type ferric cement (Sul processo di fabricazone dei cementi ferrici—tipo Ferrari)

FABIO FERRARI, *Il Cemento* (Milan), V. 47, No. 5, May 1950, pp. 82-84

Reviewed by GENNARO MIANULLI

The author points out the advantages of manufacturing ferric cements of the Ferrari type. The mixture consists of the same raw materials used to produce ordinary portland cement (lime, argillaceous limestone, marl, etc.), with the addition of iron oxide (ordinarily as pyrite ashes) in small amounts (generally about 2 percent of the mix). Silica in moderate proportions may be substituted for ferric oxide or calcareous—siliceous sand which in certain cases may entirely replace clay or marl low in calcium carbonate. Ferric cements are characterized by an Al_2O_3/Fe_2O_3 ratio of approximately 0.64. The merits of the ferric cement mixture, the manufacturing process, the saving of combustible material and production increases are discussed. The computation of the mix, consisting in the solution of the quaternary system, is also given in detail.

Structural theory

HALE SUTHERLAND and HARRY LAKE BOWMAN. Fourth Edition, John Wiley & Sons, Inc., New York. 394 pp. $5.00

Reviewed by PAUL ROGERS

This is a new edition of a well established and standard textbook on structural analysis for undergraduate courses. Certain additions have been made, like the brief explanation of the Wichert truss, of the Morris method of successive corrections and analytical establishment of the fixed points for continuous beams.

Besides the analytical methods of solving stresses and reactions, considerable space is devoted to graphic statics, emphasizing the importance and practicability of graphical solutions.

Subsequent chapters deal thoroughly with the problems of roof trusses, truss and girder bridges, influence lines, three-hinged arches, cantilever bridges, portals and lateral bracing systems, deflection of beams and trusses by several methods, classical and modern analysis of rigid frames, approximate wind stress analyses, indeterminate trusses, secondary stresses and space frameworks.

The treatment of such an extensive program in one volume is by no means an easy task and the authors are to be complimented for the clear and concise explanations which leave no doubt in the reader's mind as to the validity of the theories explored.

As a mild criticism, this reviewer believes that the graphical solution of fixed points could have been included together with their application to the solution of bending moments in continuous beams.

Fifty years of concrete and reinforced concrete in Germany (50 Jahre Beton und Stahlbeton in Deutschland)

E. BORNEMANN, *Beton u. Stahlbetonbau* (Berlin), V. 45, No. 2, Feb. 1950, pp. 31-35

Reviewed by RUDOLPH FISCHL

The economical production of concrete in Germany was started with the foundation of the German Concrete Assn. in 1898. This

paper shows how mixing, conveying and workability of concrete have been revised and changed during the years.

New prospects arose with the development of reinforced concrete. Theory, research, specifications and design in cooperation created such structures as the Century Hall in Breslau, built in 1913, with a dome 213 ft in diameter. Fifteen years later, a supermarket in Leipzig was built and the problem, to cover a square area with a span of 213 ft without interior columns, was solved by using a thin shell between arches.

The development in other branches of construction is discussed, such as highways, dams, heavy foundations, bridges and other structures. The latest development in precasting and prestressing is shown, and the author points out the influence of other countries in this rapid development, especially France and the United States. He expresses the hope, that the German engineer may have an active part in finding new ideas in the future development of reinforced concrete.

Influence lines at tenth-points for continuous beams (Zehnteilige Einflusslinien fuer durchlaufende Traeger)

G. Anger, Wilhelm Ernst & Sohn (Berlin), 1949, 6th edition, V. 1. 221 pp. 15 DM plus postage

Reviewed by Rudolph Fischl

Well-known and widely used in Europe, this work, originally in one volume, has been enlarged to three volumes in the sixth edition.

The first volume gives, for continuous beams up to eleven supports with free or fixed end conditions, for any kind of loading and equal or unequal spans, ready formulas for the moments over the supports, the reactions at the supports and the shear forces. A brief introduction shows the derivation of the moment formulas from the theorem of the three-moment equation.

It takes considerable time and effort to determine the moments of a complicated case by means of these ready formulas. There are too many auxiliary values to figure and to put into too long formulas, thus increasing the possiblity of errors. The same result can be obtained much easier and more comprehensively by the Cross method.

However, this first volume contains the fundamental formulas from which the tables for practical use in the announced second and third volumes are figured. These enlarged tables should be, judging from the previous one-volume edition, really timesaving in the design of reinforced concrete structures and bridges, and an asset for every structural library.

New Danish concrete and reinforced concrete specifications (Dansk Standard 411)

Dansk Ingenioerforening (Copenhagen), June 1949, 60 pp.

Reviewed by Eivind Hognestad

These new specifications, which were issued by the Danish Society of Engineers and later accepted as Danish Standard, apply to normal reinforced concrete structures such as buildings, bridges, hydraulic structures and smoke stacks.

The specifications contain five sections: general specifications, and special consideration of bridges, buildings, hydraulic structures, and smoke stacks.

The general specifications for materials, proportioning and construction are similar to most other specifications. It may be noted, however, that a fatigue test in which stress varies 2 million cycles from 50 to 90 percent of the yield point is specified for all reinforcement. Furthermore, no hooks are required for deformed bars.

The chapter pertaining to design and allowable stresses is unique in several respects. The analysis of monolithic structures may be made on the basis of elasticity, plasticity, or estimated restraints. Dimensioning is generally referred to the "Standard Theory"; even the dimensioning of centrically loaded columns is based on elasticity. For centrically loaded tied columns it may be noted that the allowable load is a function of the spacing and the volume of ties; and for spiral columns, the allowable load increases with the amount of spiral reinforcement. It is further interesting to note that the allowable tensile stress in hard grade deformed reinforcement is specified over 30,000 psi.

Design curves for footings on soil

Winfred A. McCracken, *Proceedings* ASCE ,V. 76, Separate No. 35, 1950

Many design tables for footings have been published, but where such tables are comprehensive, they become cumbersome, and,

because of this cumbersomeness, are difficult to use.

A study of the Code of the American Concrete Institute (ACI 318-47) shows that the depth-width ratio and the reinforcing are controlled (a) in footings of uniform thickness by bond, and (b) in sloped, or stepped footings by bond or shear, or both. It also shows that the depth-width ratio for plain concrete footings is controlled by tension on a section at the face of a coumn.

The method presented is more flexible and less cumbersome than any method involving tables, and it shows the relation between the code requirements clearly. For reinforced footings, only three sets of curves containing seven variables need to be used. For plain concrete footings, only one set of curves with three variables is required.

Following a derivation of equations, the curves are explained and an example of their use is given to make the procedure clear. This section is followed by a brief explanation of the adaptation of the curves to rectangular, one-way and continuous footings.

With the exceptions, as explained under "Derivation of Equations for Curves," the ACI code is used throughout.

Are our buildings rationally designed? (Er vore bygninger rationelt dimensionerede)

N. M. PLUM, *Ingenioeren* (Copenhagen), V. 59, No. 22, June 3, 1950, pp. 454-462

AUTHOR'S SUMMARY

The economy of a structure is today commonly judged only by its initial costs. This is very erroneous. It is the combined initial and maintenance costs which should be minimum. In a number of cases this judgment will lead to better construction with higher initial costs which, however, gives much smaller maintenance costs and a saving in the combined total costs.

With regard to floor slabs in dwellings and offices the public demand on safety first and at all costs has led to the opposite combination of too high initial costs and too low repair costs. This paper shows that the total combined costs of initial and maintenance costs will be minimum when the margin of safety is reduced about 35 percent from a present nominal value of about 4 to about 2.6. At the same time the number of collapsing floor slabs will increase from a very negligible figure to 110-425 a year. Speaking mainly about reinforced concrete floors, it is, however, estimated that these damages will not involve the risk of human lives.

The author further questions the magnitude of the live loads stated in the building codes to be used for the calculation of slabs, etc., and after a comparison of the live loads prescribed in many countries, which comparison gives very little information regarding the reasons for the requirements which seem to be very uniform, a proposal for actual determination of live loads in dwellings is given.

New all-concrete railroad station in the Netherlands (Het nieuwe station Enschede)

H. G. J. SCHELLING, *De Ingenieur* (Netherlands), Oct. 13, 1950, pp. 51-58

Reviewed by J. W. T. VAN ERP

This new railroad station is constructed entirely of reinforced concrete, including not only the structure itself but floors, walls, partitions as well, the majority in prefabricated units. The architect aimed at complete unity of material throughout the building, in line with the school of thought of the French architect Auguste Perret, whose all-concrete structures still are outstanding examples of functional as well as modern architecture. In its century of existence, concrete has been rarely used for architectural reasons only and it is therefore worthwhile to note a structure in which concrete has been used without any plaster camouflage or painting whatsoever. And nowhere in the finishing of the concrete have attempts been made to imitate stone or other materials. In the prefabricated parts surfaces have been finished off by sandblasting and various effects have been obtained by using aggregates or admixes of coloring matter, always keeping in mind the permanence of construction. An interesting feature is the use of concrete shells as form work which remain permanently in place, becoming an integral part of the structural (poured) members. These prefabricated shells are about 3 x 6 ft and 1¾ in. thick.

This method of construction called for greater precision than is ordinarily needed in building construction. It was necessary to fabricate parts to close tolerances, this in turn facilitating assembly and speed of

erection. In many parts the tolerance was about $\frac{1}{16}$ in. The building was designed on a modular grid of 17.5 x 17.5 ft.

Swedish state concrete specifications, Part 1 —Materials (Statliga betongbestaemmelser)

Statens Offentliga Utredningar 1949: 64, Department of Communications, Stockholm, 1950, 72 pp. 1.50 Sw. Kr.
Reviewed by EIVIND HOGNESTAD

These specifications were issued by the following government bureaus: Roads and Hydraulic Structures; Railroads; Hydraulic Power; Building Construction; Telegraph; and the State Institute of Materials Testing. The specifications apply to all concrete structures constructed by the institutions mentioned, at their expense or under their control. It is planned to issue the specifications in two parts: 1. Materials, and 2. Construction. The first part consists of nine chapters.

The first four chapters deal with specifications for the component materials cement, aggregates, water and reinforcement. It may be noted that yield points of deformed bars are specified over 50,000 psi only.

The following four chapters pertain to construction and control, testing of concrete, load tests, and general considerations of reinforcement.

The final chapter specifies allowable stresses. The general concepts of design refer to elasticity. Plastic formulas are specified, however, for centrically loaded columns and columns with small eccentricities. It is interesting to note that the allowable stresses for hard grade deformed bars are of the order of 30,000 psi and that the allowable steel stresses in general depend on whether the loads have a static or a dynamic character. The allowable stresses for steel as well as concrete further depend on the construction methods and the strictness of controls.

It is felt that specification-writing bodies in this country may profit by studying these standards in detail.

Concrete specifications in various countries (Betongbestaemmelser i nagra olika laender)

LAGE THEMNER, *Cement och Betong* (Malmoe), V. 25, No. 1, Mar. 1950, pp. 49-60
Reviewed by EIVIND HOGNESTAD

This article, written on the occasion of the introduction of new Swedish specifications, presents a comparison of the Scandinavian, German and ACI Standards for concrete and reinforced concrete. The following standards are considered: Denmark—1949, Finland—1946, Norway—1939, Sweden—1949, Germany—1943, and ACI—1947.

Such major considerations as component materials, qualities of reinforcement, concrete qualities, concrete classes, minimum cement contents, reinforcement details, and allowable stresses are discussed briefly. It is found that the various specifications are fairly similar, a major difference being that the "steel country" U. S. A. is behind Europe in the efficient use of high grade reinforcement.

Practical statics (Praktische statik)

RUDOLF SALIGER, Franz Deuticke, Vienna
Reviewed by PAUL ROGERS

The literal translation of the title of this book is misleading, because the content is far more extensive than practical statics would imply. In fact, this well known European textbook is a combination of structural theory, analysis, design, steel and reinforced concrete, etc. Its nearly 700 pages contain very extensive information on practically every phase of structural engineering.

The first chapter is devoted to the engineering fundamentals. Moments, shears, force polygons, analytical and graphic statics, strength of materials, water and earth pressures are thoroughly explained.

The second chapter deals comprehensively with compression and tension members, conventional and ultimate design of reinforced concrete members, the analysis of reinforced concrete beams as tied arches, combined bending and axial loadings, the establishment of the second kern as it is applicable to the design of brick chimneys, dowels for timber girders, deformations, buckling, torsion, two-way reinforced concrete slabs, flat-slabs and bearings.

In the third chapter thorough analyses are given for beams with simple and fixed-end supports, continuous beams, influence lines and alternate loadings.

One hundred and ten pages are devoted in the fourth chapter to the analysis of arches, domes and shell structures, while

the fifth chapter deals with the analysis of rigid frames including the moment distribution system.

The sixth chapter includes the analysis of trusses, both simple and statically indeterminate, and of the hinged portal frames for vertical and horizontal loads.

The book contains 660 excellent illustrations, which are self-explanatory. It is regrettable that the author did not use the international nomenclature.

Handbook of experimental stress analysis

M. Hetenyi, Editor-in-Chief, John Wiley and Sons, New York; Chapman and Hall, London, 1950. 1077 pp. $15.00

Reviewed by Douglas McHenry

One might quote from the publisher's announcement, "A unified presentation of all existing experimental methods for the determination of mechanical strength, written by 31 top rank stress analysts," which is a fair appraisal since it exaggerates only to about the extent that it underrates this voluminous compilation by Hetényi and his collaborators. ".... all existing" overstates the case only moderately, while "mechanical strength" conveys a very incomplete idea of the content of the book. Chapter headings, of which there are 18, include such items as testing machines, electric-inductance gages, electric-capacitance gages, motion measurements, residual stresses, structural model analysis, analogies. The three appendices cover fundamentals of the theory of elasticity, dimensional analysis, and the precision of measurements. A principal virtue of the book is that it is somewhat more up-to-the-minute than one might reasonably expect of a work covering a field in which advances are being made so rapidly; another is that for a composite work of 31 authors it is exceedingly well organized. The bibliographies which supplement each chapter have been made particularly useful by reviewing briefly the contents of most of the references. Members of the ACI may be disappointed that within the 1000 odd-pages only a few lines are devoted explicitly to plain or reinforced concrete, but perhaps that points up a weakness in the concrete research fraternity rather than in the book. A work of this nature has been needed for a long time, and this one fills the need in excellent fashion. It will be very useful to all who are engaged in materials testing, structural model testing, observations of static and dynamic behavior of machines or structures, and related fields.

The rigid frame bridge

Arthur G. Hayden and Maurice Barron, John Wiley and Sons, Inc., N. Y., $5.00

Reviewed by Alfred Parme

This third edition modernizes a book that in the past has been a valuable reference for the designers of rigid frame bridges. The authors have made many revisions, clarifying obscure passages and introducing noteworthy simplifications in the design calculations. The book commences with a concise presentation of the principles involved in the design of indeterminate structures. The application of these principles is demonstrated by complete design calculations of several typical examples embracing single- and double-span rigid frame bridges in both right angle and skew layouts. In the examples and in a special chapter consideration is given to the many practical factors involved such as the effects of footing restraint, lateral earth pressures, sidesway, rib shortening, shrinkage and creep. The discussion of these incorporate the experience gained in the design of many rigid frames and continued observation of existing structures.

Of particular significance is the new material devoted to the determination of influence lines by Maxwell's theorem of reciprocal deflections. The chapter on skew bridges has beem completely rewritten with much new information contributing to a better understanding of this three-dimensional problem. The design method given in this chapter reduces the analysis of a skew rigid frame to that of a comparable right angle frame modified by a few easily determined correcting factors. The effect of skew is isolated thus enabling the authors to determine a critical skew angle. Bridges with a skew less than this critical value require no skew analysis. It is regretable that in view of the controversial nature of the validity of the basic assumptions used in the analysis of skew bridges, that more substantiating data was not included.

Two complete new chapters on research on rigid frame bridges regarding the stress

concentration at junction of leg and deck, and architecture of short span bridges round out the wealth of detailed information contained in the book.

An introduction to experimental stress analysis

GEORGE H. LEE. John Wiley and Sons, New York, 1950. 319 pp. $5.50

Reviewed by GLENN MURPHY

The author introduces his subject with a review of the development of experimental techniques in supplementing analytical procedures of stress analysis, and emphasizes the interdependence of the two methods. In the first three chapters, comprising about one quarter of the book, the analytical basis of strain analysis is outlined. The equations for principal stresses are developed at the beginning of the first chapter, followed by the differential equations of elasticity for the two-dimensional and three-dimensional stress situations, the Airy stress function and the Saint Venant torsion theory. Chapter 2, on stress determinations from measured strains is a down-to-earth treatment of analytical graphical and nomographic methods of converting data from strain rosettes to principal stresses.

In the chapter on structural similitude the author does not generate much enthusiasm for models, and utilizes the first two pages in enumerating the disadvantages of models. His initial approach to the design conditions is open to serious question and can be misleading because of arbitrarily selected relationships between model and prototype that may not be valid in a specific case. The more general approach following is based on dimensional analysis and is entirely sound. The reviewer disagrees with the author's statement that dimensional analysis is limited in application to geometrically similar structures.

The description of testing equipment contained in the following 50 pages is an excellent summary of mechanical, optical, acoustical, and electrical gages. In general, there is given the type of information in which a prospective user would be interested. The principles of operation, the advantages, disadvantages and applications are well presented. The chapter on resistance wire strain gages, written as an introduction to the topic, is an effective survey of the possibilities and limitations of this type of gage for both static and dynamic strains.

Photoelasticity as an experimental tool is discussed in two chapters; one, on theory and equipment; the other, on the experimental procedures involved, from the selection of the material to the principal stress determination at any point. Both two-dimensional and three-dimensional techniques are described.

In Chapter 8, Dr. Lee discusses the use of a few of the analogies available for solving the Laplace and the Poisson equations. The membrane analogy for the solution of torsion problems is described in some detail, but does not include the use of the calibration circle. Brief discussions of the conventional electrical and hydrodynamical procedures for solution of the Laplace equation complete the chapter, which is a brief introduction rather than an authoritative treatment of this phase of experimental stress analysis.

A brief treatment of the brittle lacquer method is followed by a chapter on miscellaneous methods including brittle materials, photogrid, and Beggs deformeter. Unfortunately, the author has cited fatigue testing as a means of evaluating stress concentration factors, ignoring the fundamental difference between stress concentration and endurance limit reduction.

The final chapter includes consideration of the selection of the appropriate experimental method for a given problem, interpretation of strain gage readings, gage calibration, error, and a group of six laboratory problems for class use.

With the exception of the chapters on theory of elasticity and analogies, mathematics beyond calculus is not required, and much of the remainder is descriptive and comparative rather than analytical. For the engineer seeking an introduction to the experimental method, the book has much to offer. It is well organized, clearly written, and contains excellent illustrations and drawings. In addition, it is thoroughly documented with references well chosen for the reader who wishes to pursue any branch of the subject further.